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Wie-Kong Jie
San Jose State University

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Jie, Wie-Kong, M.S.

San Jose State University, 1993

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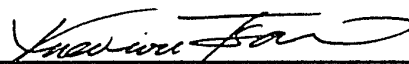
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Presented to
The Faculty of the Department of
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In Partial Fulfillment
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Master of Science

By
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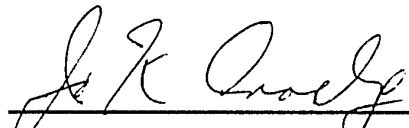
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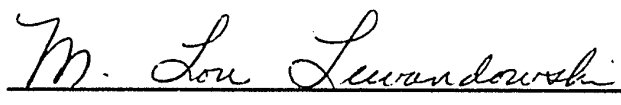
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ABSTRACT

ESTIMATION OF PILE FRICTIONAL CAPACITY IN CLAYEY SEDIMENTARY DEPOSITS IN TAIPEI USING CPT DATA

by Wie-Kong Jie

This thesis demonstrates that the present state-of-the-art methods for utilizing Cone Penetration Test data for pile frictional capacity prediction in cohesive soils agree reasonably well the observed maximum capacity of the tested piles when using methods suggested by Tumay and Fakhroo (1981). However, the method suggested by Schmertmann (1978) tends to underestimate the load carrying ability of the piles.

The observed maximum frictional capacity is defined from the load versus butt settlement curve of pile test results.

The difference between these two computed frictional capacities is due to the significant difference of their relevant adhesion factors for cone friction lower than 0.36 kg/cm^2 , where the adhesion factor from Tumay et.al. is higher than from Schmertmann.

AUTHOR'S NOTE

The six instrumented test piles and eighteen case studies presented in Chapters 3 and 4, respectively, are from geotechnical projects in which the author participated as engineer for Moh and Associates, Inc., (MAA) of Taipei, Taiwan. The information and many of the figures are from project reports and other documentation; because this information is proprietary, the instrumented data and cases are identified by letter and number designations only, sources are referenced as MAA project reports, and the actual reports are not listed in Appendix A. References. Appreciation is extended to MAA and to the individual contributors who provided access to these materials.

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The author is particularly indebted to his advisor, professor Kuei-Wu Tsai for his understanding, patience, guidance, and inspiration. Great appreciation is extended to Professors Jo Crosby and T.C. Kao for serving as members of the thesis committee as well as for giving useful suggestions and advice.

Grateful thanks are extended to the author's colleagues for their interest and support. L. S. Lin and I. S. Wang did the drafting of all the figures. Research assistance was provided by Louis Shu and C.C. Huang, and word processing help was given by Catherine Liao.

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CHAPTER I

INTRODUCTION

The determination of pile bearing capacity from in-situ penetration test results (Standard Penetration Test and Cone Penetration Test) using empirical approaches, has been widely used. The problem of estimating pile capacity, however, is complicated by the soil characteristics, pile types and installation procedures. The effect of these influencing factors is difficult to predict by analytical techniques due to the complex phenomenon of soil-pile interaction. Load tests are helpful in verifying the estimated pile capacity.

Recently, large diameter cast-in-place bored piles constructed with the reverse circulation drilling technique have increasingly been used in Taipei, primarily due to their relatively low cost, low noise and low vibration levels.

The objective of this thesis is to compare the loading capacity of axially loaded piles, as indicated by full-scale field load tests, to predicted frictional capacity which contributes the major portion of the load carrying capacity of

piles installed in cohesive soils, utilizing the results of the Cone Penetration Test. A series of eighteen cast-in-place reverse circulation bored piles located in Taipei Basin, where subsurface conditions consist of clayey sedimentary deposits, were loaded. These loading values are then compared to the predicted values utilizing the results of the Cone Penetration Test performed at the sites, following the method suggested by Schmertmann (1978), and Tumay and Fakhroo (1981). This thesis demonstrates that the present state-of-the-art methods for utilizing Cone Penetration Test data for pile frictional capacity prediction in cohesive soils agree reasonably well with the observed carrying capacity of the tested piles when using the method suggested by Tumay et.al. However, the method suggested by Schmertmann tends to underestimate the load carrying ability of the piles in this region.

CHAPTER II

REVIEW OF THE LITERATURE

The basis of the soil mechanics approach to calculating the carrying capacity of an axially loaded pile is that the total resistance of the pile to compression loads is the sum of two components, namely skin friction and end resistance. A pile bearing on rock or some other hard incompressible material is known as an end-bearing pile (Figure 2.1a), while a pile in which the skin-frictional component predominates is known as a friction pile (Figure 2.1b).

For an end bearing pile, the total end bearing capacity can be expressed as

$$Q_p = A_p q_p$$

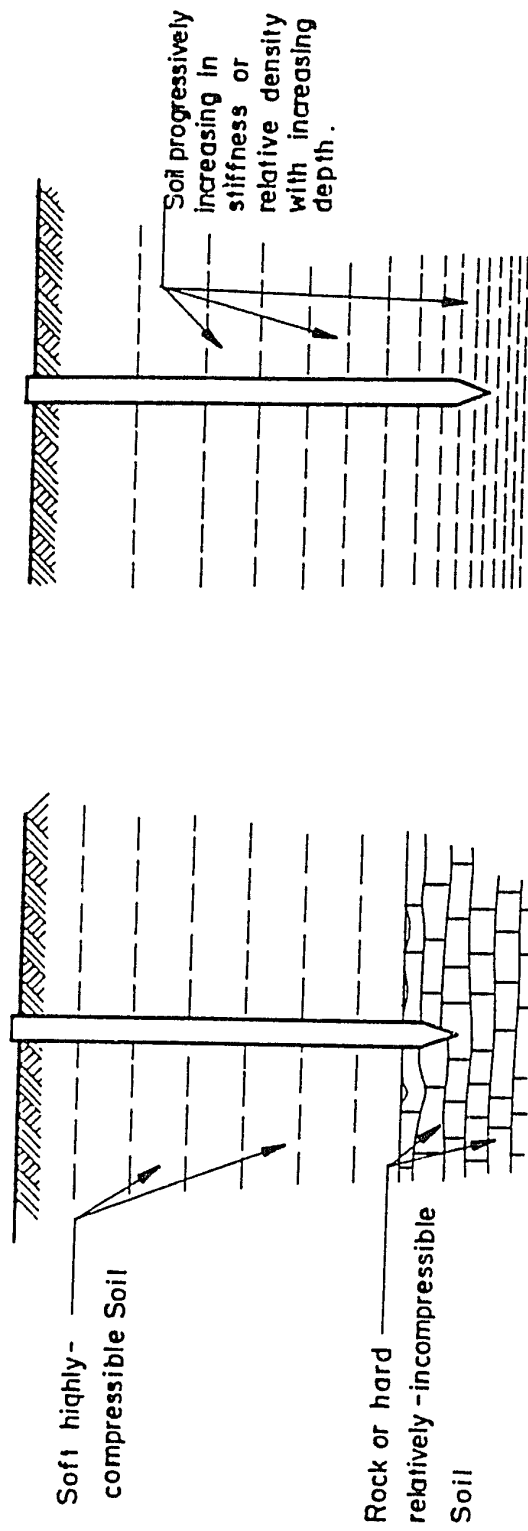
where

$$Q_p = \text{total end bearing capacity}$$

$$A_p = \text{area of pile cross section}$$

$$q_p = \text{unit end resistance}$$

While for a friction pile in cohesive soils, the total



(a) End - bearing pile

(b) Friction pile

Figure 2.1 Types of Bearing Pile (Redrawn After Tomlinson, 1977)

shaft adhesion, Q_s acting on a pile may be expressed as

$$Q_s = \pi D \int_0^L f \, dL$$

where D = pile diameter
 L = pile length
 f = unit shaft adhesion
 dL = length of a pile element

In the present state of knowledge, the effect of the various methods of pile installation on the carrying capacity and deformation characteristics can hardly be calculated by the strict application of soil or rock mechanics theory. One of the ways that it can be done is to apply simple empirical factors to the shear strength of the undisturbed soil or to the in-situ test parameters. These empirical factors are generally dependent on the particular method of installation and are based on experience and on the results of field loading tests.

The unit shaft adhesion, f in cohesive soils is normally predicted using a total stress approach in which the undrained shear strength of the clay, S_u is multiplied by an empirical adhesion factor α , i.e.

$$f = \alpha S_u$$

Table 2.1 Adhesion Factor for Bored-and-cast-in-situ Piles in Clay (after Tomlinson, 1977)

| Location | Type of clay | Undrained cohesion, KN/m^2 | Adhesion factor | Reference |
|-------------------|------------------------|-------------------------------------|-----------------|-------------------|
| SE England | London clay | 50-240 | 0.3-0.6*** | Skempton |
| Lutterworth, UK | Lias clay | 2.00 | 0.1-0.3* | - |
| Grimsby, UK | Glacial till | 1.40 | 0.4-0.6 | Taylor |
| Glasgow, UK | Glacial till | 1.60 | 0.85 | - |
| Glasgow, UK | Laminated glacial clay | 0.96 | 1.0-1.2 | - |
| Canada** | Keilor clay | 1.10 | 0.31 | - |
| Keilor, Australia | Kerang clay | 1.4-1.6 | 0.34-0.36 | - |
| Kerang, Australia | Beaumont clay | 1.2-1.9 | 0.46-0.28 | - |
| Houston, USA | | 0.96-1.9 | 0.40-0.52 | O'Neill and Reese |
| Lemoore, USA | | 0.96 | 0.49-0.52 | - |

* This clay was very sensitive to the addition of water. It is believed that the low adhesion factors were due to slurring by drilling tools from water in fissures. On the same site driven precast concrete piles had adhesion factors of 0.27 and 0.9.

** Location not known.

*** Observed values of skin friction did not exceed 96 kN/m^2 ($1\text{ KN/m}^2 = 0.01\text{ kg/cm}^2 = 20.89\text{ lb/ft}^2$)

Experience from pile tests has provided a variety of the adhesion factor α as shown in Table 2.1, where the value of α can vary from 0.1 to 1.2 depending on soil type and strength (Tomlinson, 1977). The most common method used by the engineers in Taiwan is that presented by Skempton, who suggested that the α value varied from 0.6 for long piles, with favorable geological conditions and careful workmanship, to 0.3 for very short piles and with unfavorable conditions, with an average value of 0.45 (Skempton, 1959).

Although the above method for shaft adhesion estimation appears straight forward it does have major difficulties. At the center of these is the problem of measuring the undrained strength, S_u . The fact is that there is no unique value of S_u for a soil element. Its value depends on many factors such as the orientation of the stresses, the rate of testing, the size of the specimen and the test method. Moreover values on S_u , whether determined in the laboratory or in the field, frequently show a wide scatter.

Many investigators now believe that the cone frictional resistance, f_s , data obtained from the Cone Penetration Test provides a better indication of the undrained shear strength, S_u of cohesion, f . Experimental correlation of f_s and S_u presented by Wesley (1967) indicated f_s to be slightly higher than S_u . Begemann (1965) and Vesic (1967) suggested that f_s may be taken as equal to or half of the unit shaft adhesion, f of piles,

respectively. Nottingham (1975) applied Tomlinson's adhesion factor, α to relate f to average f_s values. He assumed undrained cohesion, S_u to be equal to f_s .

In this thesis, the intent is to use the cone frictional resistance, f_s , data for directly estimating the unit shaft adhesion f , in cohesive soils. Two experimental correlations are compared:

- 1) α' method suggested by Schmertmann (1978), and
- 2) Cone-m Method suggested by Tumay and Fakhroo (1981).

Schmertmann's Method

The computation of shaft adhesion in cohesive soils using the mechanical Cone Penetration Test data, as described by Schmertmann, follows the relationship expressed as

$$F_s = \alpha' f_s A_s$$

where F_s = total ultimate shaft friction
 α' = ratio of pile to penetrometer sleeve friction in clay, as shown in Figure 2.2
 f_s = average undrained sleeve friction of cone

A_s = total soil-pile contact area

This recommendation was based primarily on an evaluation of the results from the model and full-scale pile studies done by Nottingham in 1975.

Tumay and Fakhroo's Method

The same relationship between shaft adhesion and average cone friction was suggested by Tumay and Fakhroo as

$$F_s = m f_s A_s$$

where m = adherence coefficient, as shown in
Figure 2.3

The correlation in Figure 2.3 was obtained based on field load tests of 37 test piles of various shapes, size and material, as shown in Table 2.2, driven in soft cohesive soils with the mean S_u ranging from 0.21 kg/cm² to 1.1 kg/cm² throughout 10 sites in Louisiana.

The cone penetration tests were conducted utilizing the Fugro electronic cone penetrometer with a cone of 60 degree point angle and 10 square centimeters base area penetration at

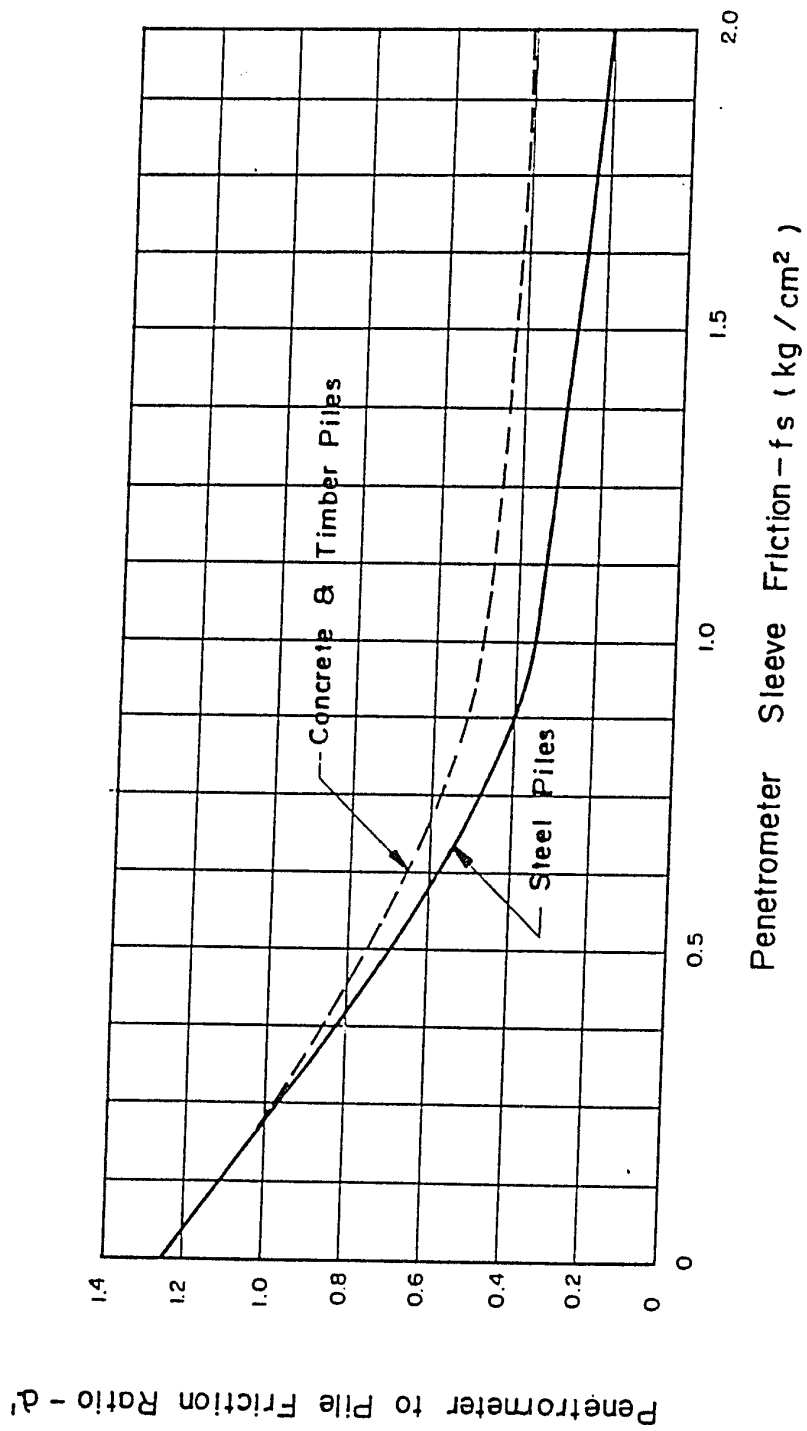


Figure 2.2 Design Curves for Shaft Friction in Clay Soils (Redrawn After Schmertmann, 1978)

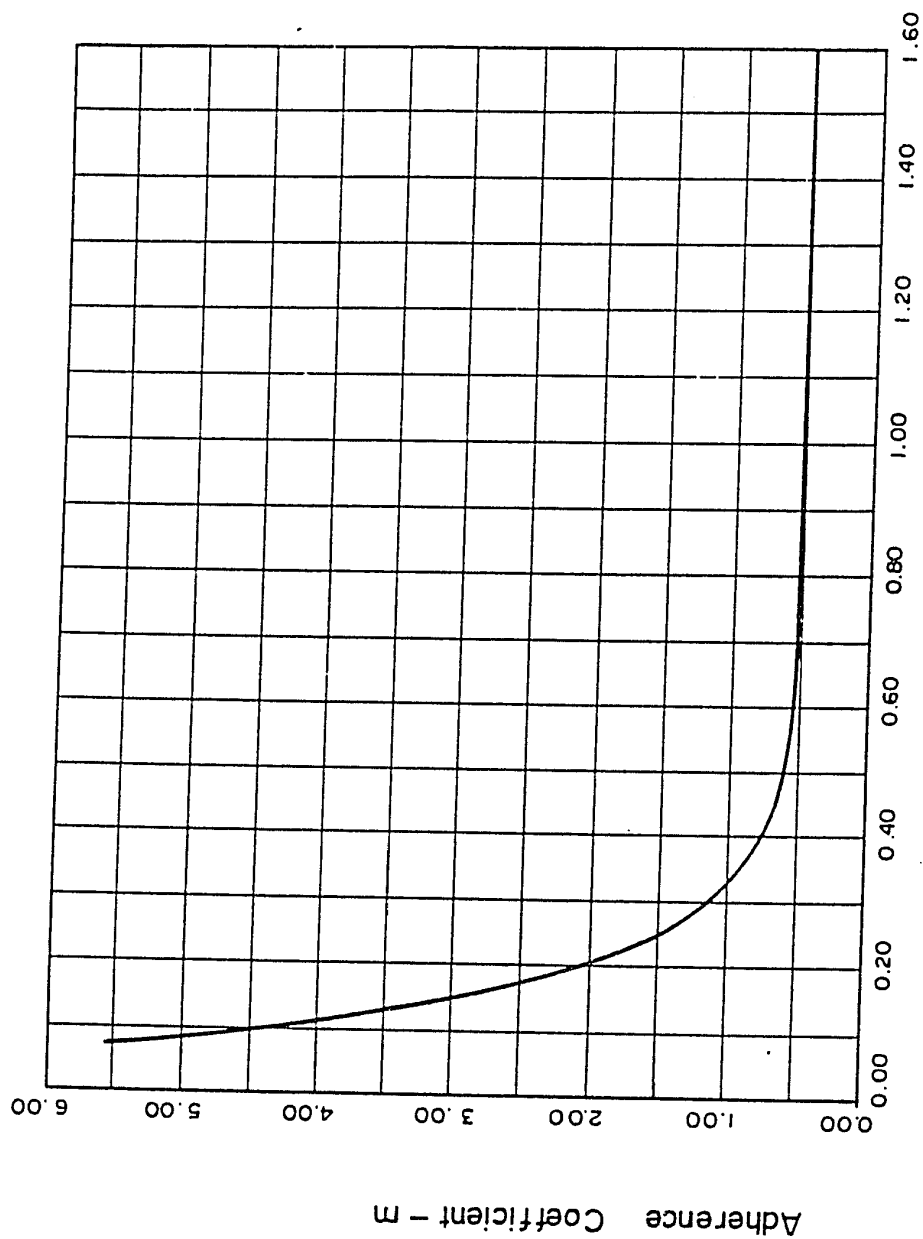


Figure 2.3 Average Frictional Resistance, \bar{f}_s , versus Adherence Coefficient, m (Redrawn After Tumay et. al., 1981)

the rate of 2 cm/sec.

According to Tumay et. al., the data points shown in Figure 2.3 were the average values of the 10 sites, and the individual test pile data fitted the m curve very closely. The algebraic mean of the prediction error (predicted-measured)/measured was reported to be -2.0% with standard deviation of 11.5%. Tumay et. al. based on the curve fitting process indicated that the m curve could be expressed by the equation:

$$m = 10.0 - 9.5 (1 - e^{-9.0fs})$$

The above expression suggested that the adherence coefficient, m, could vary between 10.0 and 0.50. Tumay et. al. suggested an upper limit of 0.75 kg/cm² for the unit pile friction, f.

Table 2.2 Test Piles Data of Tumay and Fakhroo
(after Tumay et. al., 1981)

| No. | Test Pile | Location | Shape | Embedded Length (ft) | Ultimate Load (Q) (Ton) |
|-----|-----------|-------------------|-------------------|----------------------|-------------------------|
| 1 | 1 | New Orleans (I) | 18" Square (C) | 120 | 345 |
| 2 | 2A | New Orleans (I) | Step Taper (S) | 123 | 190 |
| 3 | 3 | New Orleans (I) | Monotube (S) | 123 | 275 |
| 4 | 4 | New Orleans (I) | Step Taper (S) | 123 | 200 |
| 5 | 5 | New Orleans (I) | 16" Dia. Pipe (S) | 123 | 320 |
| 6 | 6 | New Orleans (IIA) | 16" Square (C) | 100 | 225 |
| 7 | 8 | New Orleans (IIA) | 20" Triangle (C) | 100 | 240 |
| 8 | 9 | New Orleans (IIA) | Step Taper (S) | 102 | 135 |
| 9 | 10 | New Orleans (IIA) | Step Taper (S) | 137 | 205 |
| 10 | 11 | New Orleans (IIA) | Monotube (S) | 102 | 180 |
| 11 | 12 | New Orleans (IIA) | Monotube (S) | 137 | 240 |
| 12 | 13 | New Orleans (IIA) | Step Taper (S) | 102 | 135 |
| 13 | 14 | New Orleans (IIA) | Step Taper (S) | 137 | 205 |
| 14 | 15 | New Orleans (IIA) | 14" Dia. Pipe (S) | 102 | 180 |
| 15 | 16 | New Orleans (IIA) | 16" Dia. Pipe (S) | 137 | 240 |
| 16 | 18 | New Orleans (IIA) | Step Taper (S) | 101 | 175 |
| 17 | 19 | New Orleans (IIA) | Step Taper (S) | 138 | 200 |
| 18 | 1 | Houma | 18" Square (C) | 95 | 200 |
| 19 | 2 | Houma | 12" Dia. Pipe (S) | 95 | 260 |
| 20 | 3 | Houma | 14" Dia. Pipe (S) | 95 | 130 |
| 21 | 4 | Houma | Step Taper (S) | 95 | 195 |
| 22 | 5 | Houma | Step Taper (S) | 152 | 255 |
| 23 | 10 | Houma | Monotube (S) | 95 | 175 |
| 24 | 24-1 | Ruddock | 24" Square (C) | 65 | 250 |
| 25 | 30-1 | Ruddock | 30" Square (C) | 65 | 330 |
| 26 | 24-2 | Ruddock | 24" Square (C) | 90 | 380 |
| 27 | 30-2 | Ruddock | 30" Square (C) | 80 | 600 |
| 28 | 54-2 | Ruddock | 54" Round (C) | 83 | 700 |
| 29 | 4A | Morgan City | Step Taper (S) | 120 | 180 |
| 30 | 4B | Morgan City | Step Taper (S) | 108 | 170 |
| 31 | 2A | Baton Rouge (I) | 14" Square (C) | 45 | 170 |
| 32 | 4A | Baton Rouge (IIA) | 14" Square (C) | 43 | 165 |
| 33 | 1 | Alexandria | 14" Square (C) | 31 | 105 |
| 34 | 3 | Alexandria | 14" Square (C) | 45 | 140 |
| 35 | 2 | Borgne (I) | 36" Square (C) | 124 | 500 |
| 36 | 4 | Borgne (II) | 36" Square (C) | 130 | 520 |
| 37 | 5 | Borgne (II) | 24" Square (C) | 104 | 320 |

Note: 1. (C) means Concrete

(S) means Steel

2. 1ft = 0.3048 meter, 1 Ton = 2205 lbs.

CHAPTER III

INTERPRETIVE TECHNIQUE AND ASSUMPTIONS

3.1 General

Load-displacement curves obtained from axial compression tests on deep foundations can exhibit any one of the three shapes shown in Figure 3.1. The peak of A and the asymptote of B give the maximum resistance of the foundation. However, if the load-displacement curve resembles C, the maximum resistance of the foundation is difficult to evaluate. Drilled shaft data often resemble C (Hirany and Kulhawy, 1989).

In this thesis, the intent is to define the maximum shaft resistance of bored piles based on the amount of pile displacement from pile load test data.

3.2 Previous Literature Data

Dimensionless side load versus mean settlement relationship

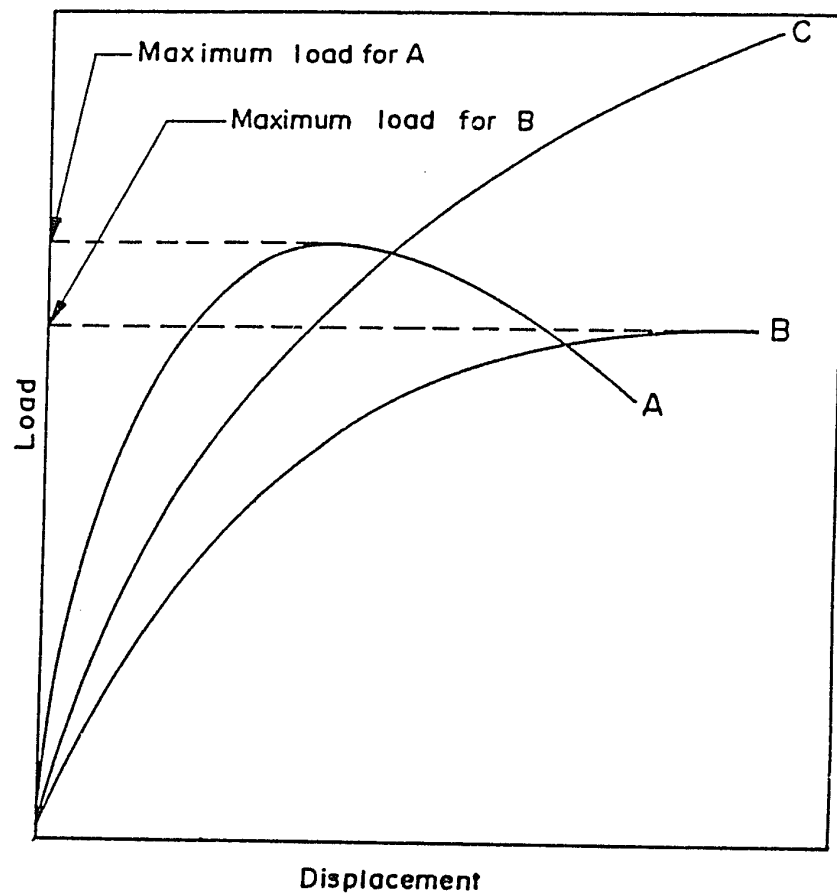


Figure 3.1 Typical Load-Displacement Curves (After Hirany et. al., 1989)

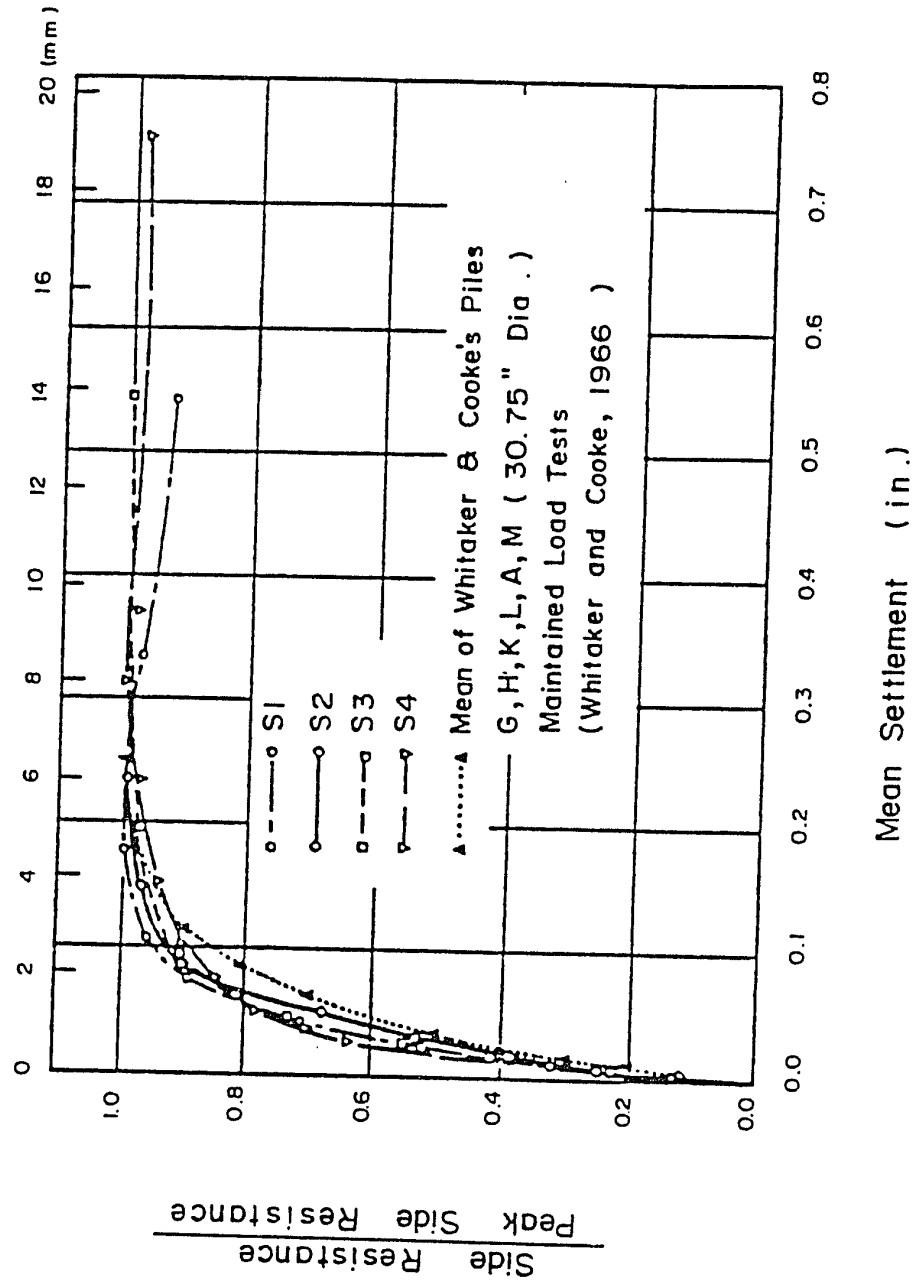
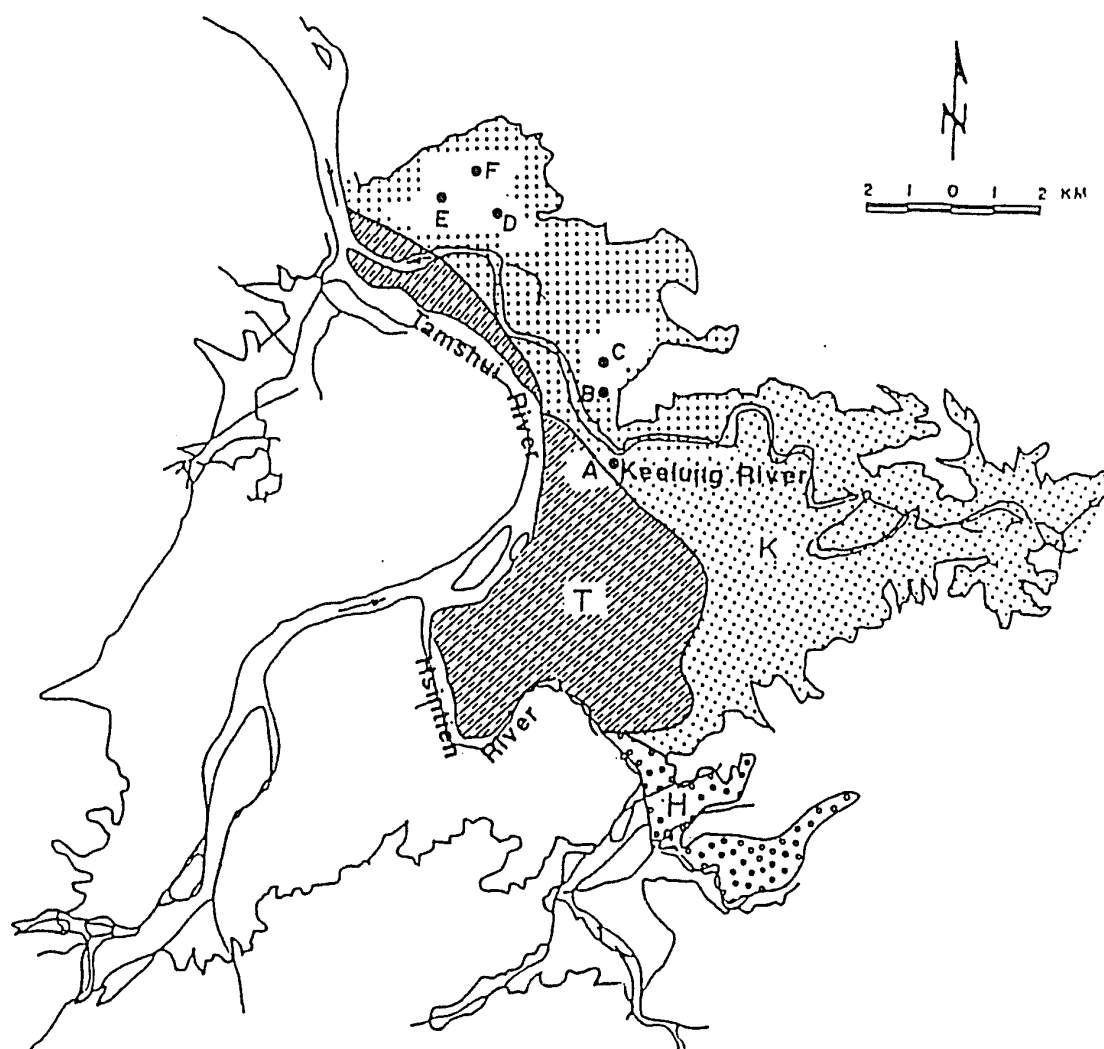


Figure 3.2 Dimensionless Side Load-Settlement Relationships
(Redrawn After O'Neill et. al., 1972)

is presented in Figure 3.2 on the basis of large scale tests on bored piles reported by O'Neill and Reese (1972). In this figure, mean settlement, i.e., the average of butt and base settlement was used; it was reported that very little concrete compression existed, and that the butt and base settlements were nearly equal.

As can be seen, Figure 3.2 shows that the slip to develop maximum skin resistance is on the order of 5 to 10 mm; Bowles (1988) referred to the study by O'Neill and Reese and reported that this amount of slip was relatively independent of shaft diameter and embodiment length. Bowles also reported that the mobilization of the ultimate end resistance required an end displacement on the order of 10 to 30 percent of the base diameter for bored piles.



Note :

T: Tamshui River Zone .

H: Hsintien River Zone .

K: Keelung River Zone .

●: A~F Test Piles .

Figure 3.3 Location of Instrumented Test Piles within
Tributary Area of the Keelung River

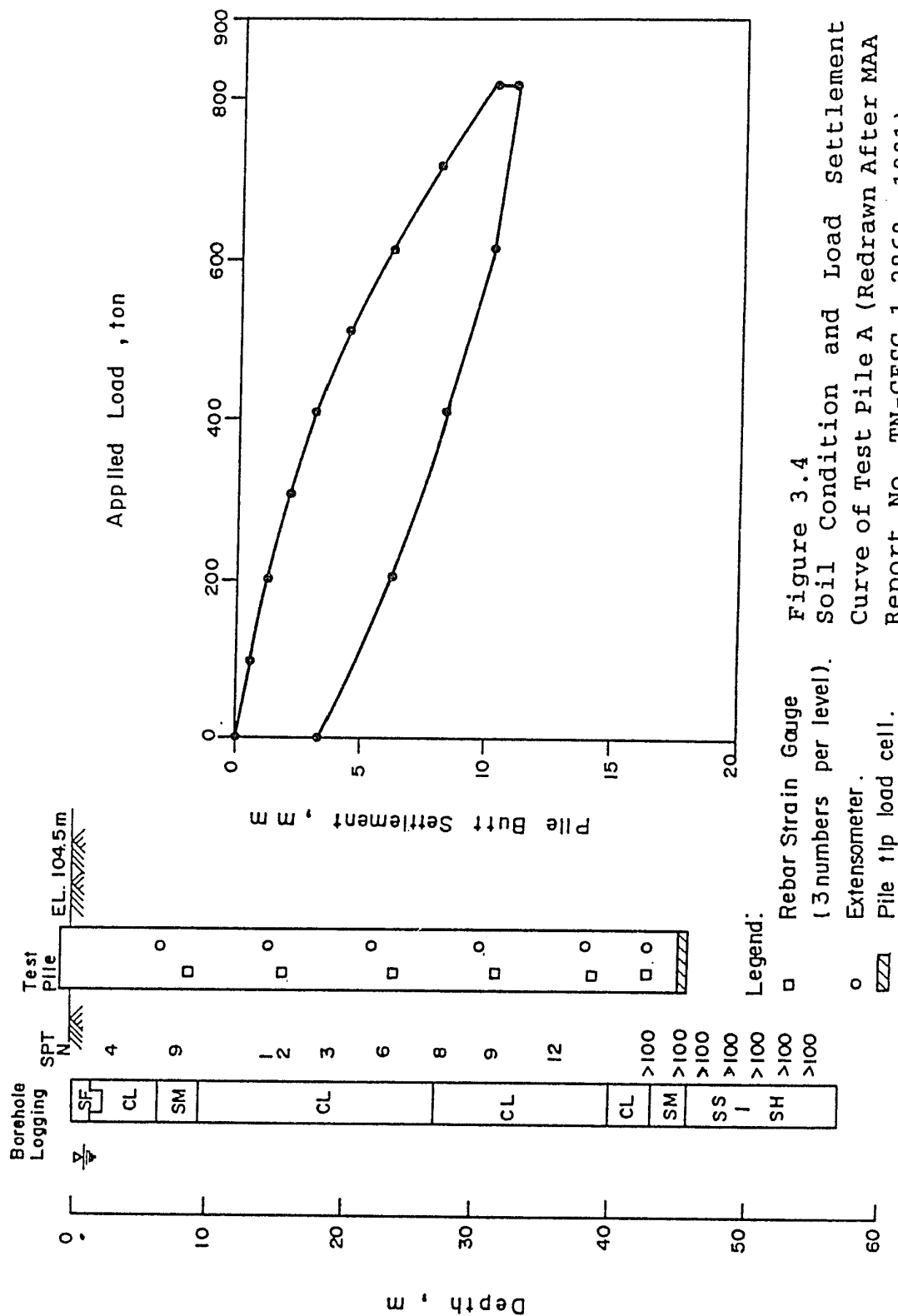


Figure 3.4
Soil Condition and Load Settlement
Curve of Test Pile A (Redrawn After MAA
Report No. TN-GESC-1-2869, 1991)

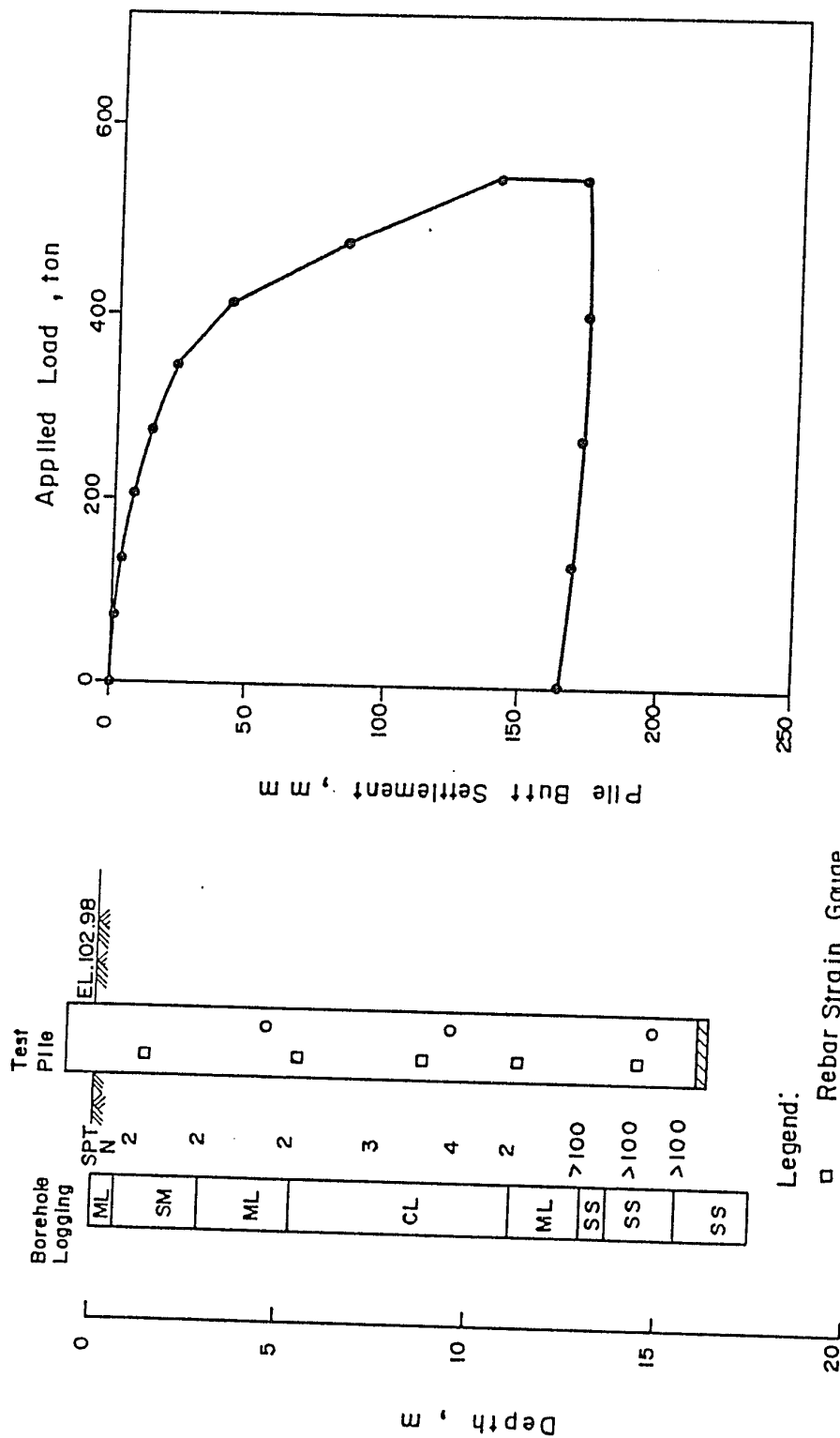
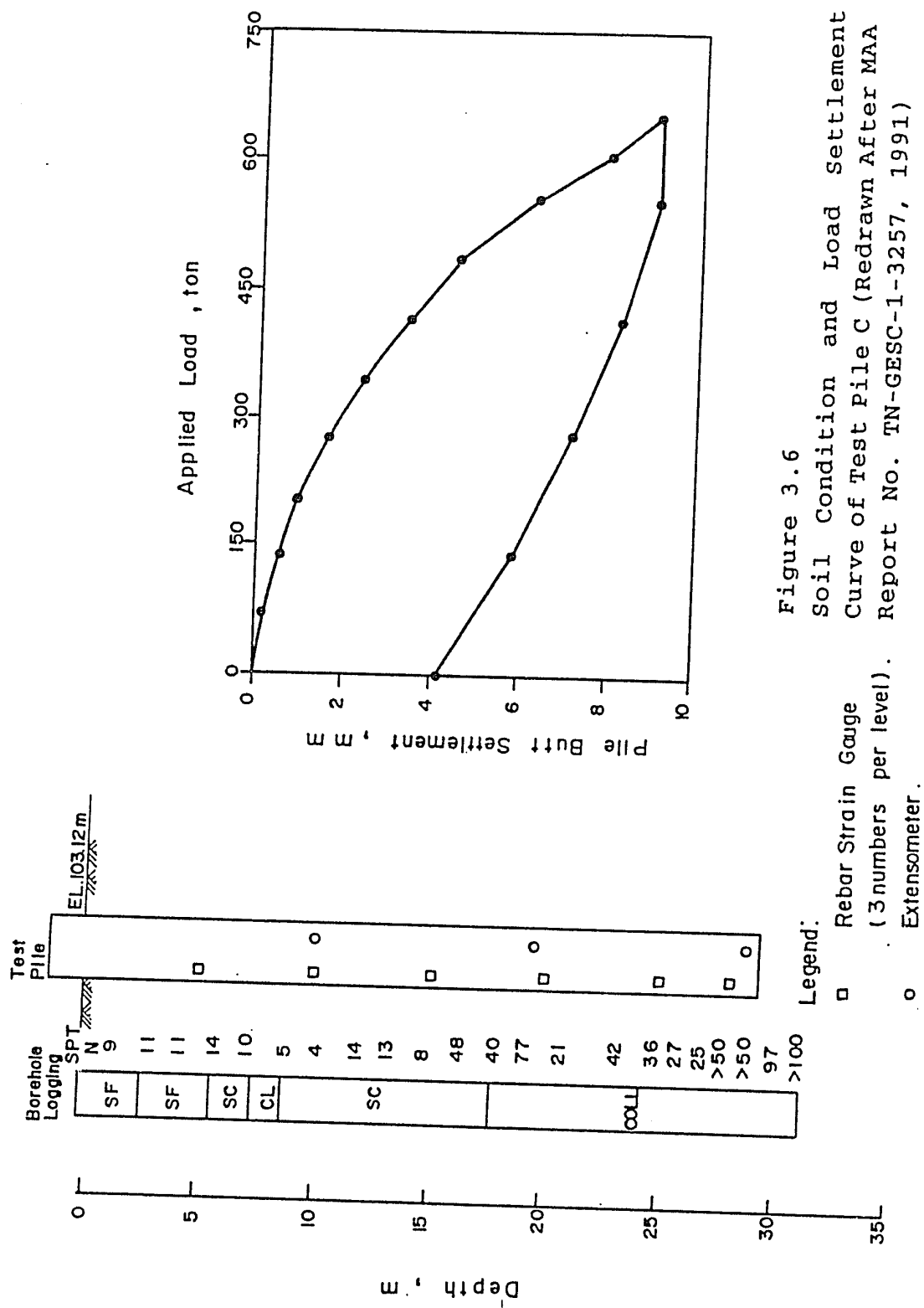


Figure 3.5
Soil Condition and Load Settlement
Curve of Test Pile B (Redrawn After MAA
Report No. TN-GESC-1-3345, 1992)



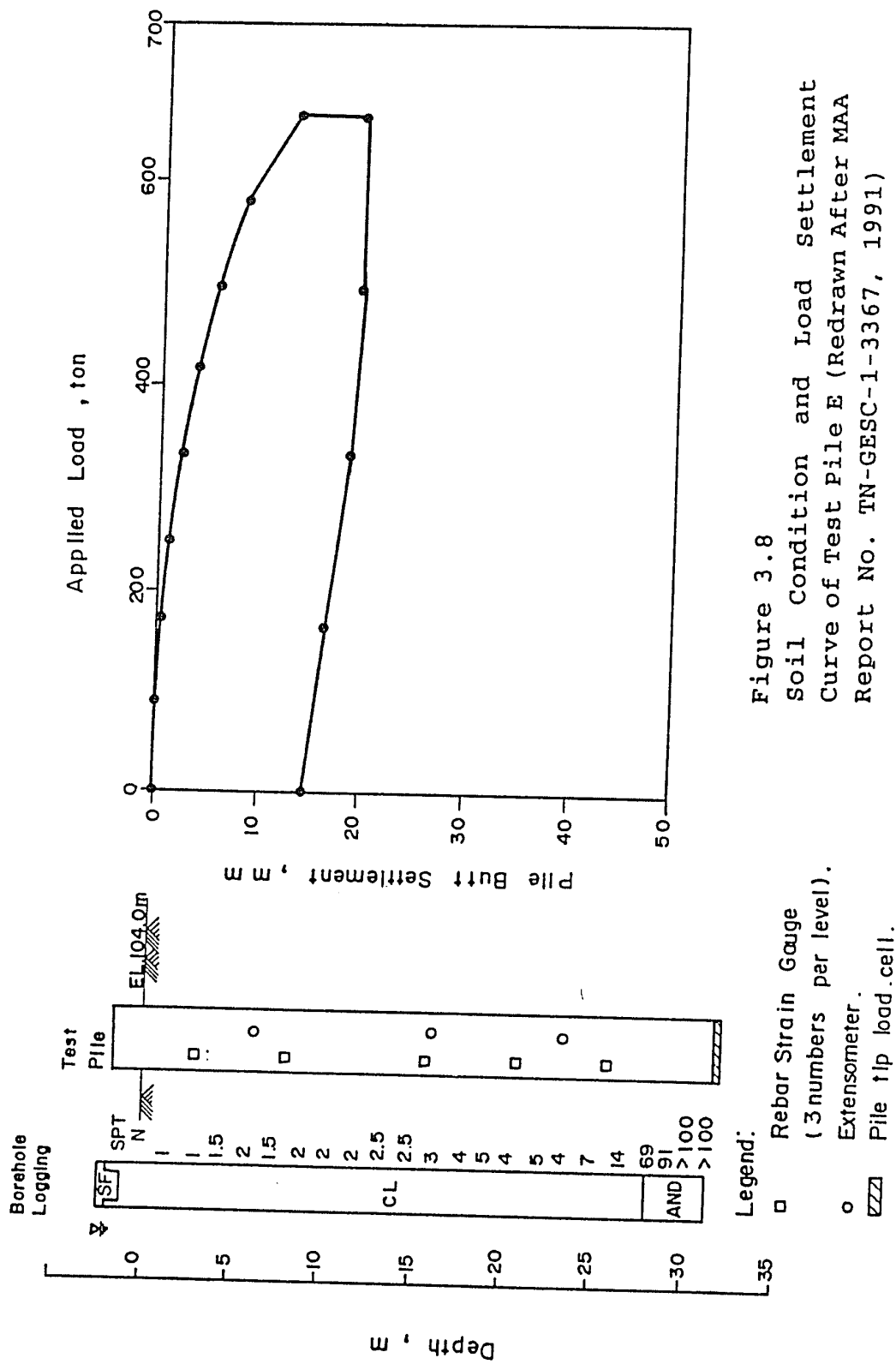
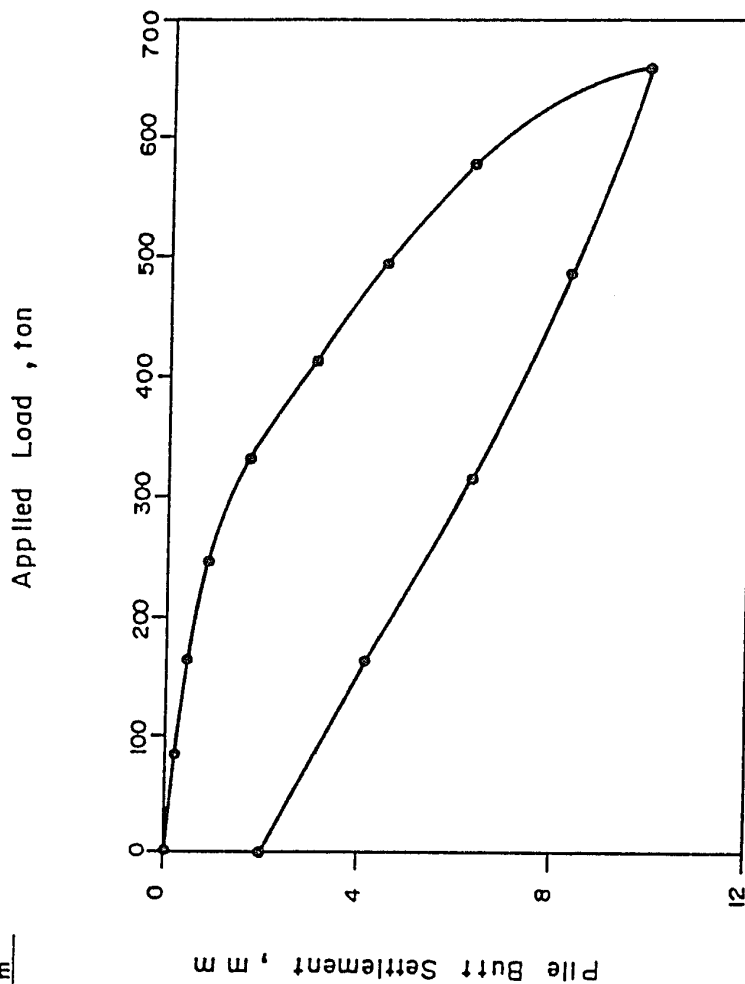
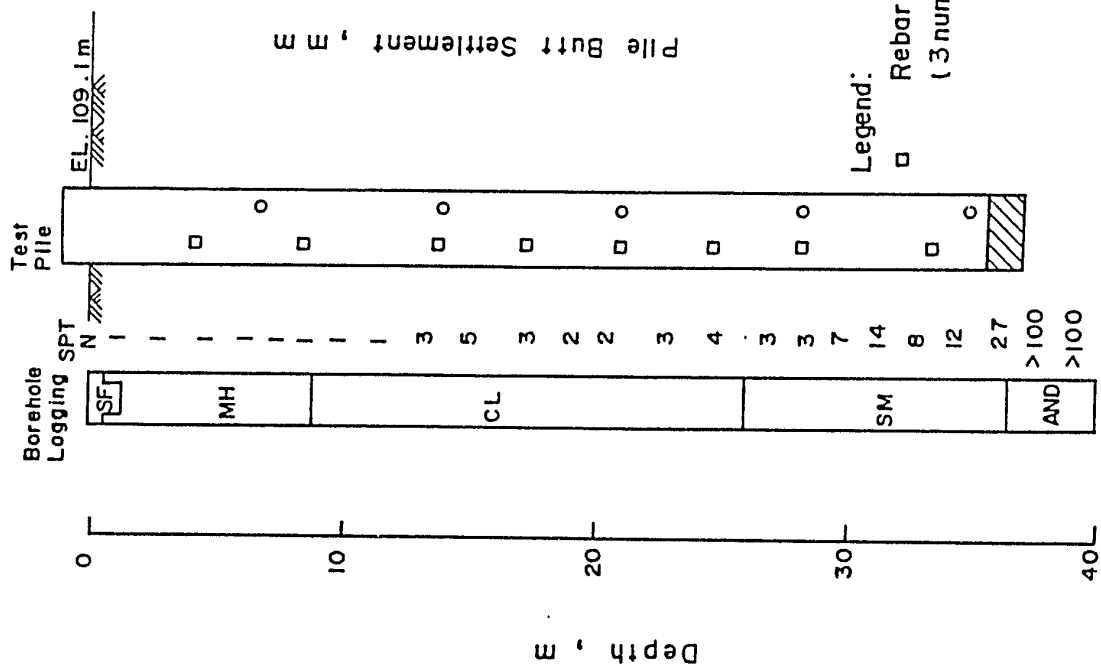


Figure 3.8
Soil Condition and Load Settlement
Curve of Test Pile E (Redrawn After MAA
Report No. TN-GESC-1-3367, 1991)



Legend:

- Rebar Strain Gauge (3 numbers per level).
- Extensometer.
- ▽ Pile tip load cell.

Figure 3.9
Soil Condition and Load Settlement
Curve of Test Pile F (Redrawn After MAA
Report No. TN-GESC-1-950, 1989)

3.3 Local Instrumented Test Piles Data

A total number of six instrumented, cast-in-situ, bored piles were tested within the tributary area of the Keelung River in Taipei Basin, as shown in Figure 3.3. The soil conditions for each location of test piles are as shown in Figures 3.4 through 3.9, with soil type abbreviations as shown in Table 3.1. Among the instrumented test pile, pile-tip-load-cells were installed at piles A, B, D, E and F. To ensure a good contact between the cylindrical load cell body and the soil, a cement grout was injected from the ground surface prior to pile concreting, through a pipe fixed in the reinforcement, into the void between load cell and the bottom of hole.

The procedure for conducting the load tests was generally in accordance with ASTM D1143, where loads were applied in eight equal increments up to a maximum test load, and then unloaded in four equal decrements. Detailed graphical presentations of the load-settlement curves are also shown in Figures 3.4 through 3.9.

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Figure 3.10 through 3.15 show the observed concrete compression of the pile-body of the instrumented test piles.

Theoretical elastic deformation for an assumed end-bearing pile can be calculated using the simple formula as

$$\Delta = PL / AE$$

where Δ = elastic deformation
 P = the applied load
 L = length of the pile
 A = cross-sectional area of the pile
 E = elastic modules of the pile

A friction pile with uniform stress distribution to the soil throughout the length of the pile will have an elastic deformation of one-half of the above. The elastic deformations for the six instrumented piles are considered as friction piles, i.e.:

$$\Delta = PL / 2AE$$

with the elastic modules of pile, $E = 2.1 \times 10^6 \text{T/m}^2$. Table 3.2 shows the calculated elastic deformations. The observed concrete compression of the instrumented test piles is also shown in Table 3.2 for comparison.

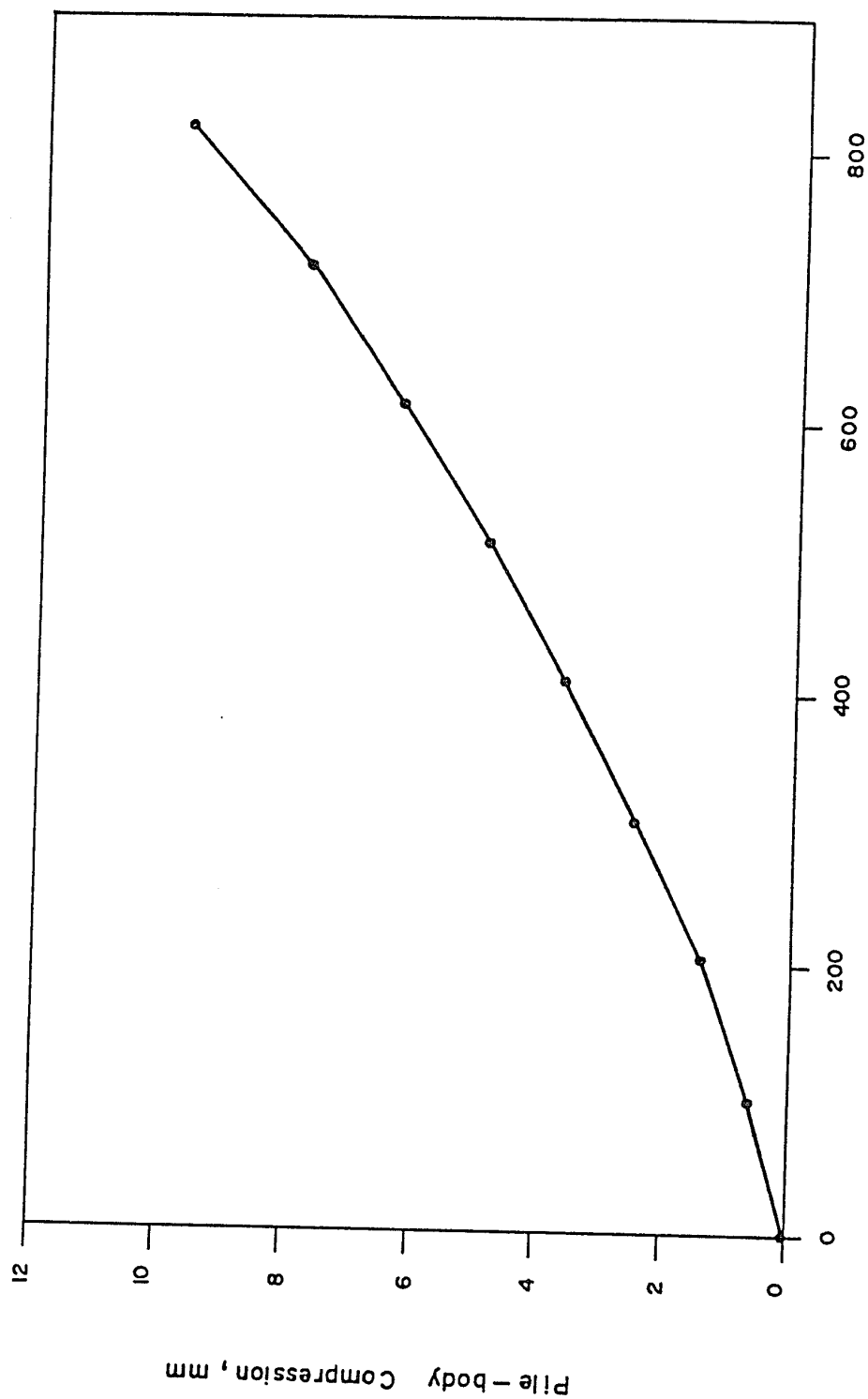


Figure 3.10 Observed Concrete Compression of Pile-body
of Test Pile A (Redrawn After MAA Report
No. TN-GESC-1-2869, 1991).

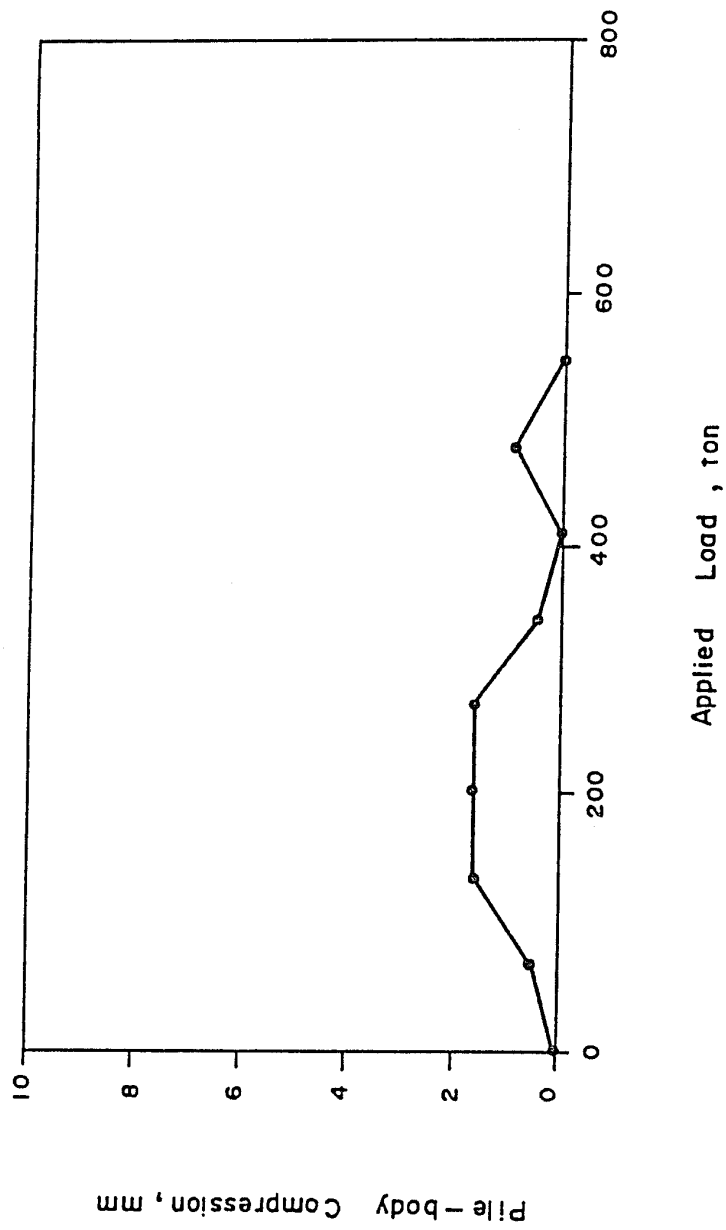


Figure 3.11 Observed Concrete Compression of Pile-body
of Test Pile B (Redrawn After MAA Report
No. TN-GESC-1-3345, 1992)

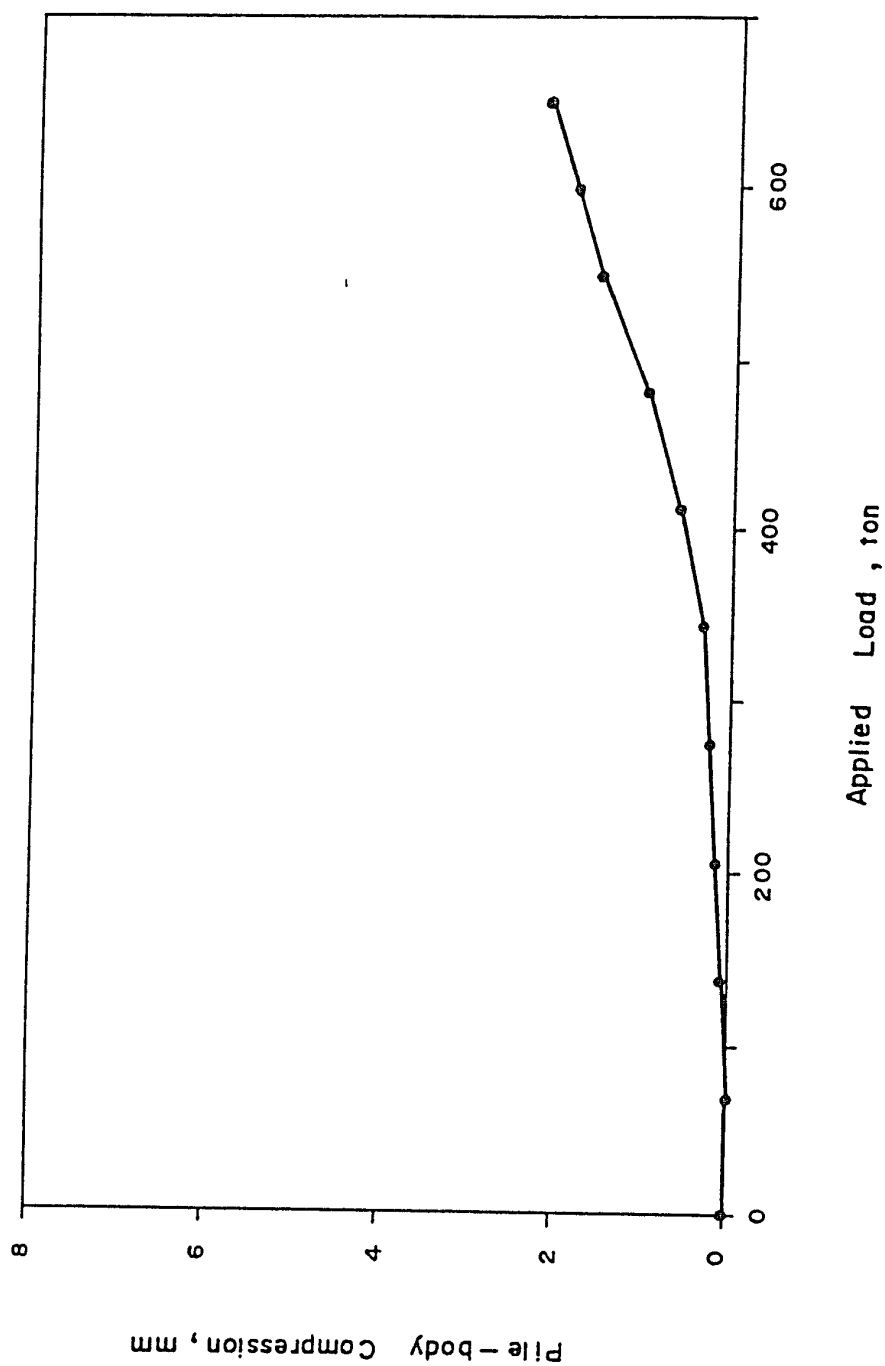


Figure 3.12 Observed Concrete Compression of Pile-body
of Test Pile C (Redrawn After MAA Report
No. TN-GESC-1-3257, 1991)

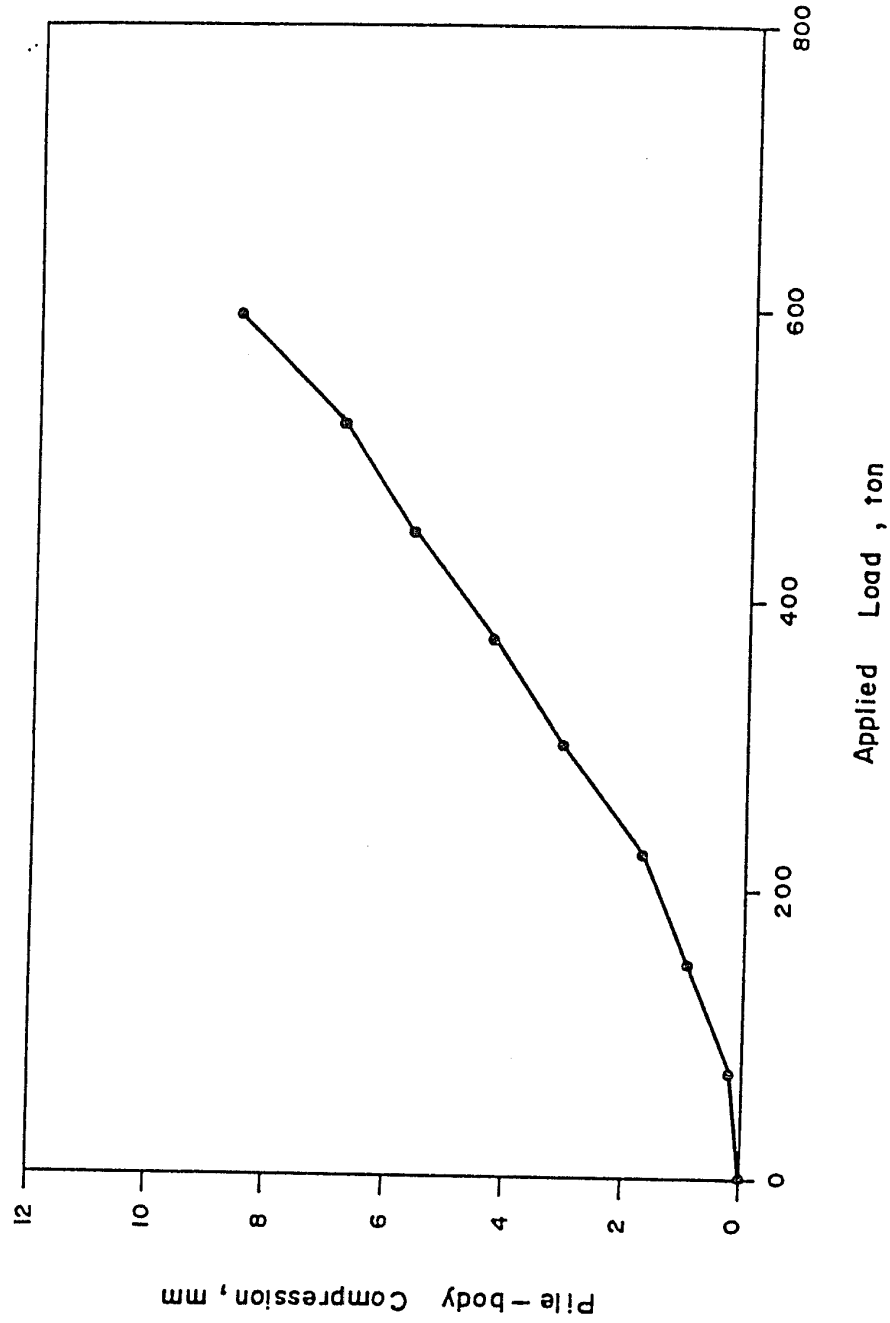


Figure 3.13 Observed Concrete Compression of Pile-body
of Test Pile D (Redrawn After MAA Report
No. TN-GESC-1-3288, 1992)

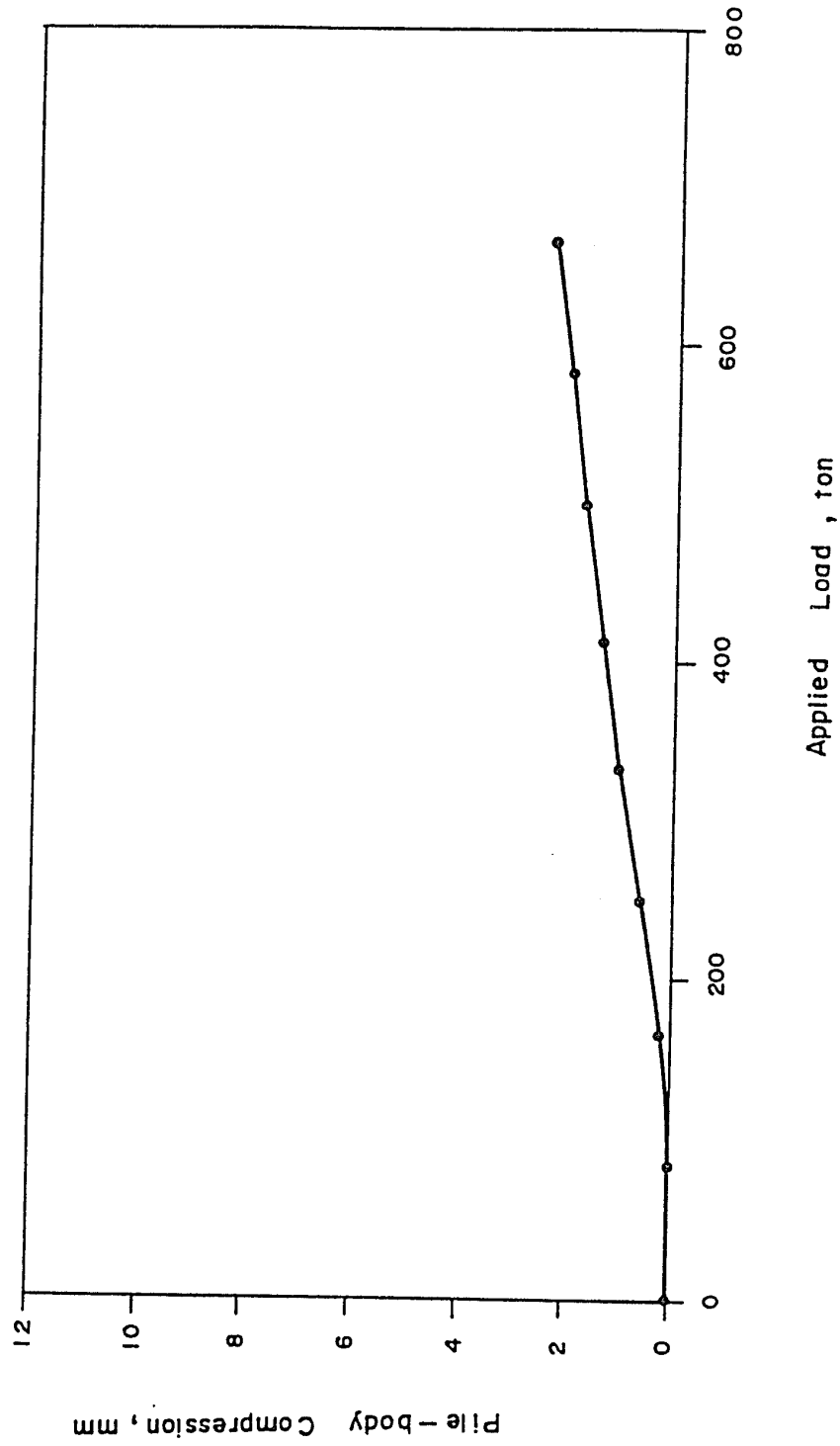


Figure 3.14 Observed Concrete Compression of Pile-body
of Test Pile E (Redrawn After MAA Report
No. TN-GESC-1-3367, 1991)

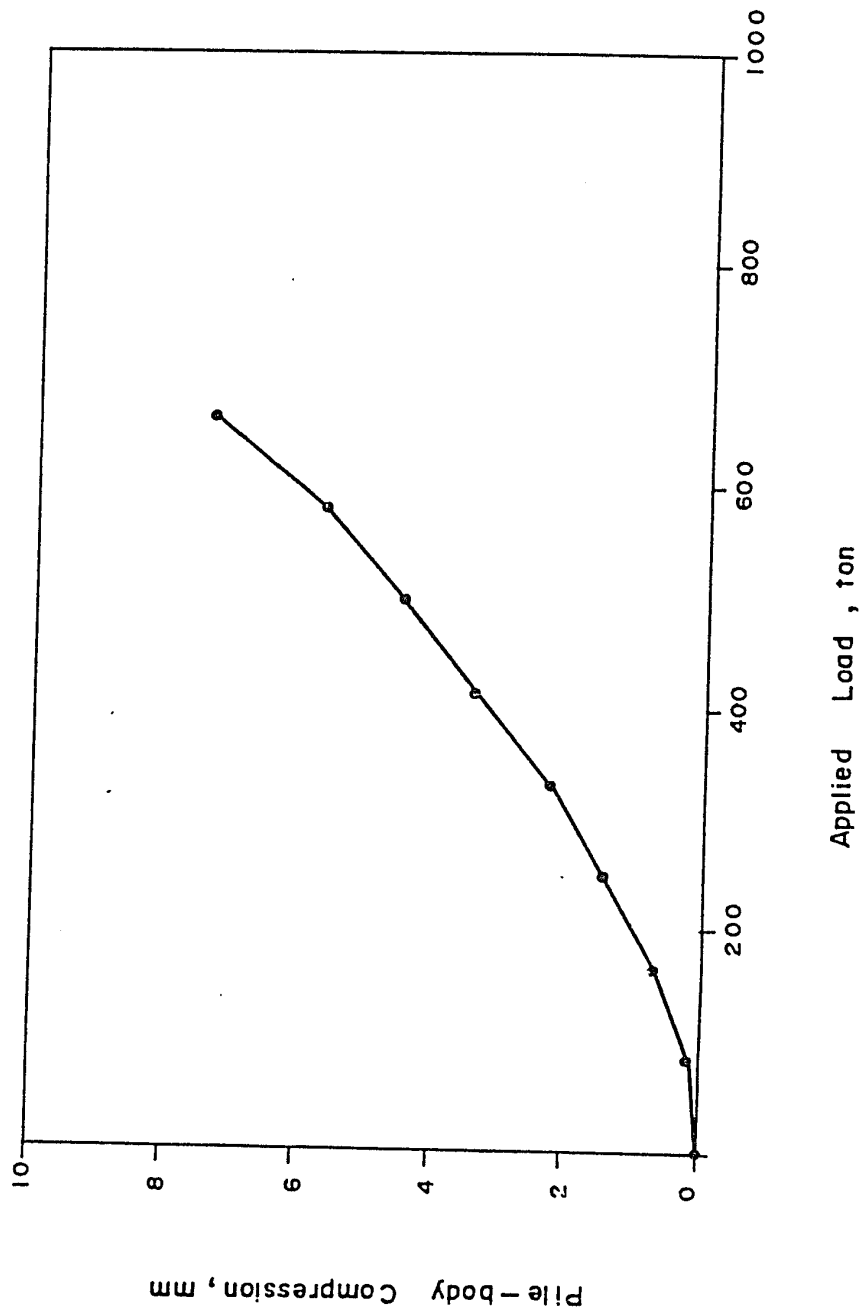


Figure 3.15 Observed Concrete Compression of Pile-body
of Test Pile F (Redrawn After MAA Report
No. TN-GESC-1-950, 1989)

As can be seen in Table 3.2, the observed concrete compressions of piles A, D and F are approximately equal to the computed elastic deformations, indicating that a good contact between bottom of the pile (i.e., load cell body) and tip bearing layer was gained by cement grouting at the pile tip executed prior pile concreting as described above. While for piles E and B, although the same grouting measures had been taken, the effectiveness was not gained. Their observed concrete compressions are nearly the same amount as that of pile C, which was constructed following the common procedure of reverse circulation drilling technique mostly used in Taiwan (as described in Chapter 4).

MAA (1992) summarized the above-mentioned six instrumented test piles data and reported that the limiting shaft friction in cohesive soil layers developed at relative slip or displacement between pile and soil on the order of 4 to 5 mm, as shown in Figure 3.16.

3.4 The Applied Interpretive Technique and Assumptions

Based on the above-mentioned information and data, it is hence assumed that for cast-in-place bored piles constructed with the reverse circulation drilling technique, the unavoidable loosening of the base of the bore and the uncleaned slime at the

Table 3.2 Calculated Elastic Deformations of
Instrumented Piles

| Pile Number | Pile Diameter, m | Pile Length, m | Maximum Test Load, Ton | Calculated Elastic Deformation (Δ), mm | Observed Concrete Compression, mm |
|----------------|---------------------|-------------------|---------------------------|---|---|
| A | 1.00 | 45.80 | 817.00 | 11.30 | 9.90 |
| B | 1.00 | 16.10 | 548.00 | 2.70 | 0.00 |
| C | 1.00 | 29.00 | 650.00 | 5.70 | 2.20 |
| D | 1.00 | 46.50 | 596.00 | 8.40 | 8.60 |
| E | 1.00 | 32.00 | 664.00 | 6.40 | 2.60 |
| F | 1.00 | 37.10 | 662.00 | 7.40 | 7.50 |

Note: 1. $\Delta = PL/2EA$

2. Elastic deformations for this thesis are calculated using $E=2.1 \times 10^4 \text{ T/m}^2$

bottom of bored holes make the butt and base settlement, at the initial loading stage where loading is resisted mostly by the shaft resistance, nearly equal.

In this thesis, the amount of butt settlement required to develop maximum shaft resistance is thus assumed to be 7.0 mm (which is the sum of 4.5 mm, a value of slip between pile and soil required to develop maximum shaft friction, and 2.5 mm, an observed value of concrete compression of the pile-body when constructed with the reverse circulation drilling technique), which is the same value as that reported by Woo and Moh (1990).

The observed maximum shaft resistance of test piles thus can be defined from the load versus butt settlement curve of each test pile, that is, the load at butt settlement is equal to 7.0 mm.

The computations of shaft adhesion in cohesive soil are based on the two empirical approaches suggested both by Schmertmann (1978), and Tumay et. al. (1981), that the unit shaft friction of the pile, f , is a function of cone friction, f_s , from Cone Penetration Test (CPT) data.

When granular soils are encountered, the relevant shaft friction is computed using the empirical formula suggested by Meyerhof (1976) as :

$$f(T/m^2) = 0.1N$$

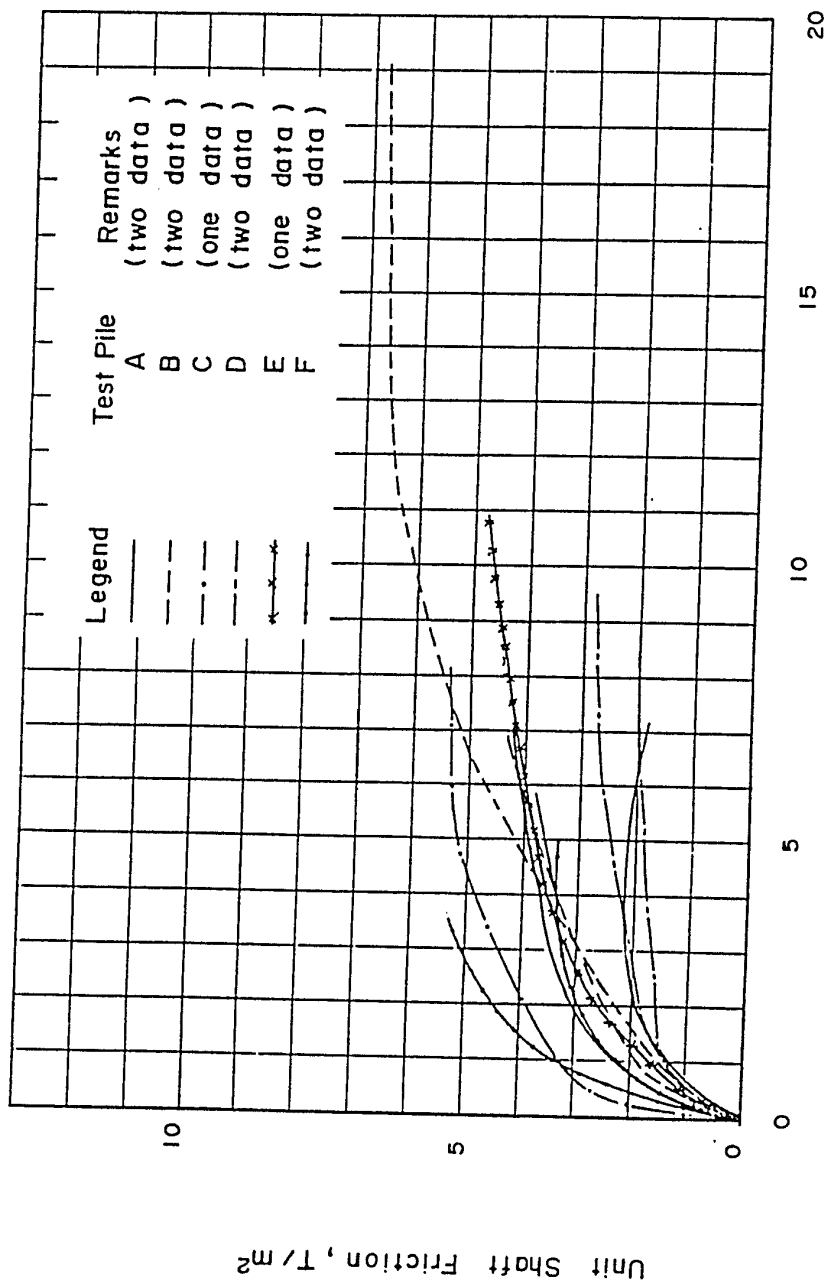


Figure 3.16 Shaft Friction-Relative Displacement Relationships in Cohesive Layers of Bored Piles in MRT
 Tamshui Line Project (Redrawn After MAA Report No. TN-GESC-1-4002, 1992)

where N = average value of Standard Penetration
Test resistance

The limiting value for shaft friction in granular soils of 12 T/m^2 , usually used by Dutch engineers, (Schmertmann, 1978) is applied in this thesis.

CHAPTER IV

CASE STUDIES

4.1 General

The gravel or rock-debris layer, with Standard Penetration Test's N-values larger than 100 in Taipei, has always been used as the bearing stratum for the bored piles. Because of the relatively deep depth of the gravel or rock-debris layer from the ground surface, long piles with pile diameters ranging from 1.0 to 2.0 m are commonly used. In normal cases, a pile is designed to take both skin friction and end bearing.

The pile load tests referred to in this thesis are those normally carried out on a cast-in-situ bored pile by the application of loading at the top of the pile with settlements at the pile top being observed as the loading is progressively increased to the full value of the test load, and also as the pile is being unloaded.

4.2 Geological and Soil Conditions

4.2.1 General Geological Setting

Taiwan is an island 180 km off the coast of the Chinese Mainland, separated by the Taiwan Straits. The island comprises about 36,800 square kilometers.

The Taipei Basin is located at the northern corner of the island of Taiwan. It lies between latitude N24°58' to N25°09' and longitude E12°25' to E12°37', occupying a total area of 243 square kilometers and with a general elevation of 9 meters above sea level (Huang, 1962). The Basin is geographically bounded by three major rivers: the Keelung River, the Tahan River and the Hsintien River as shown in Figure 4.1. Surrounding the Basin is the Tatun Volcanic Group on the North, the Linkou Tableland on the West and foothills on the East and the South.

Geologically speaking, the Taipei Basin is a tectonic basin, formed between the Pliocene and Pleistocene Age (about two to three million years ago), covered with Quaternary sedimentary deposits overlying the Tertiary bedrock. The thickness of the Sedimentary deposits is estimated, ranging from 10 meters to 240 meters, and can be divided into three formations. They are, in the order of older to younger (or from deeper to shallower): the Hsinchuang Formation (consisting of alternative layers of yellow or gray sandy clay and gravel), the

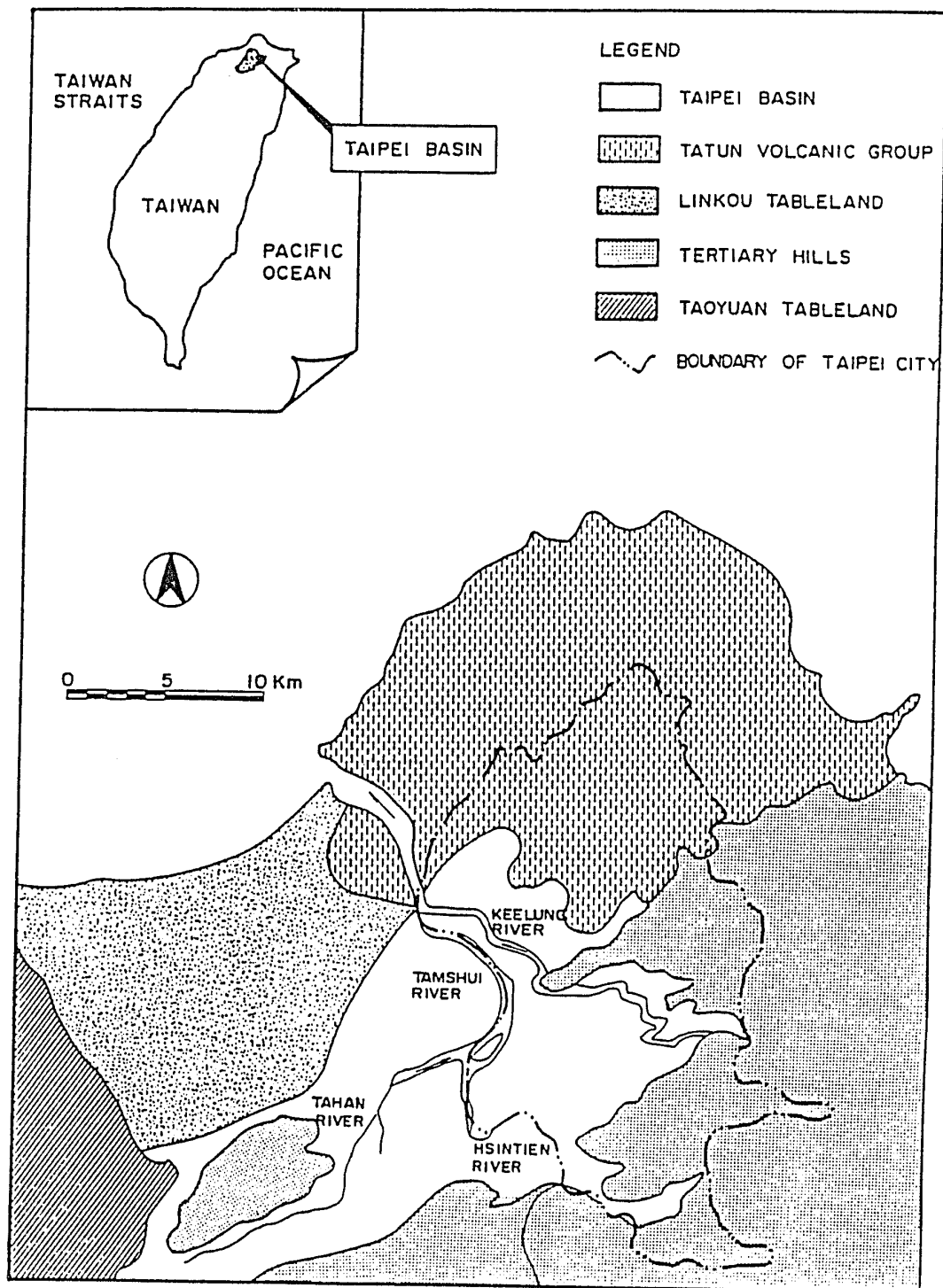


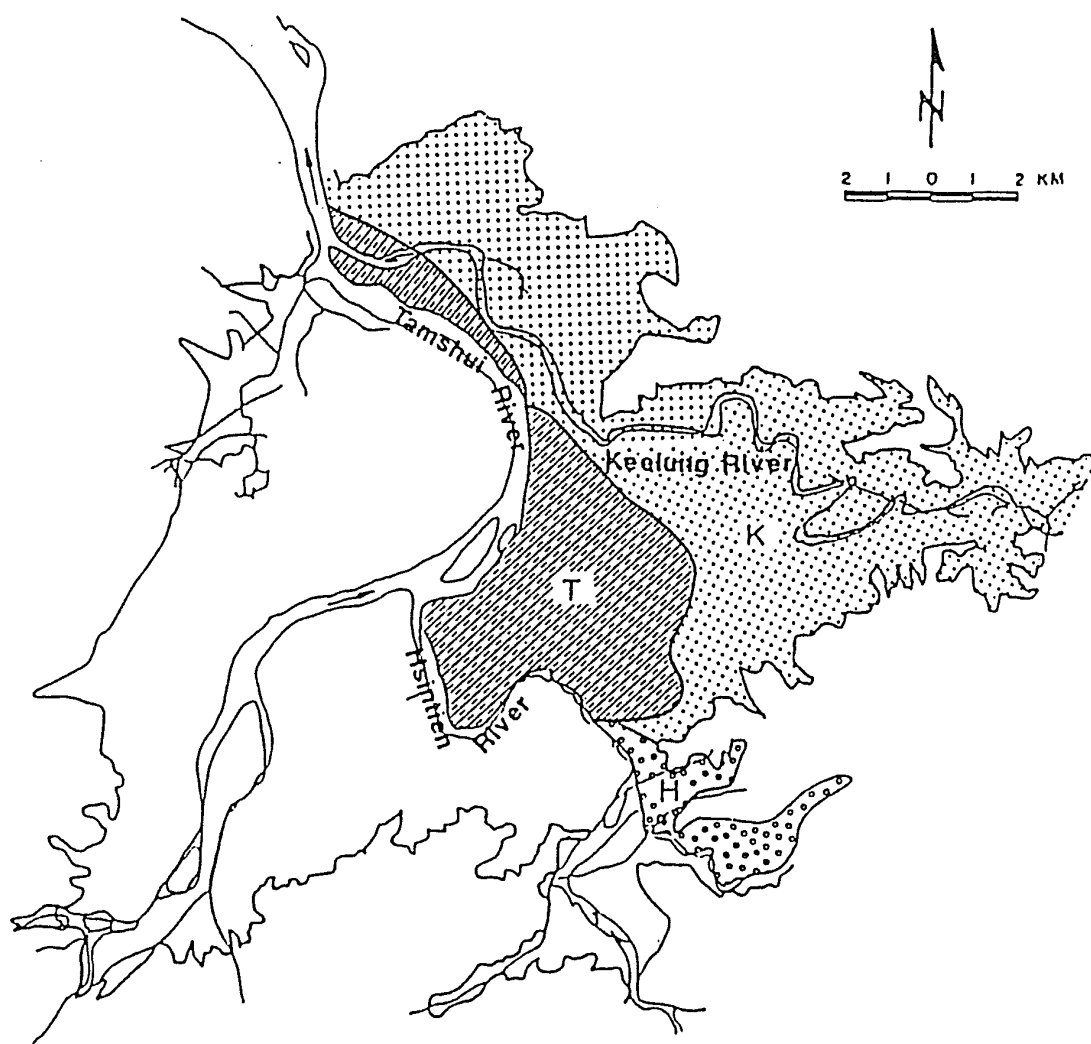
Figure 4.1 Geological Conditions of the Taipei Basin and It's Surrounding Areas

Chingmei Formation (basically a gravel layer with large amount of coarse sand) and the Sungshan Formation (consisting of alternating layers of silty clay and silts interstratified with fine sand layers containing a high silt content) (Woo and Moh, 1990).

The Sungshan Formation, which is immediately below the ground surface and extends to a depth of about 10 to 60 meters, is most significant to the underground constructions and foundations in the Basin region.

On the basis of the geological origin and sedimentary environment, the Sungshan Formation within the eastern area of the Basin was divided into three major zones according to the tributary of the rivers as shown in Figure 4.2, namely: Tamshui River Zone, Hsintien River Zone and Keelung River Zone (Hung, 1966). The sediments of the Tamshui River tributary area have the typical characteristics of the Sungshan Formation, recognized by most, and found in the downtown area of Taipei, that is alternating layers of fine sand and silt / clay. The sediments along the tributary area of the Hsintien River are predominately of sands, silty sands and sandy gravels. Whereas the basin sediments along the tributary area of the Keelung River are predominately of fine grained silts and clays.

The pile load tests referred to in this thesis are all located within the tributary area of the Keelung River.



Note :

T: Tamshui River Zone .

H: Hsintien River Zone .

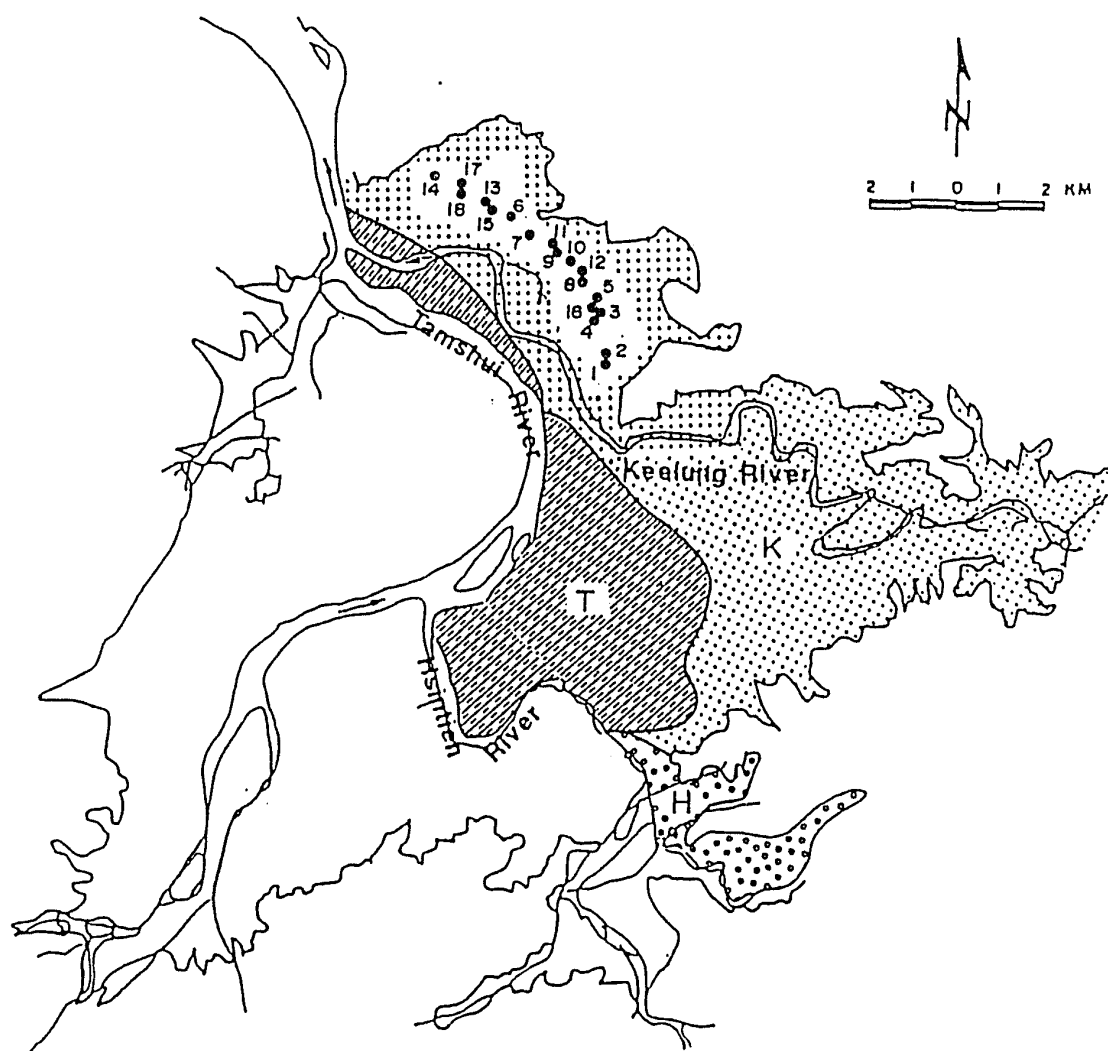
K: Keelung River Zone .

Figure 4.2 Location of Major Zones of Sungshan Formation

4.2.2 Soil Condition

A total number of eighteen piles are considered in this thesis as shown in Figure 4.3. All of them are located within the tributary area of the Keelung River, i.e., predominately consisting of fine grained silts and clays. The soil conditions for each location of test piles are as shown in Figures 4.4 through 4.21, with soil type abbreviations as shown in Table 3.1. The boreholes with Standard Penetration Test were performed within 10 meters from each test pile location, while the locations of Cone Penetration Tests (CPT) were within 50 meters from each test pile.

The soil parameters for each test pile, as shown in Figures 4.4 through 4.21, are based on the nearest reference boreholes and CPT data from each test pile, and are also based on the construction records of each test pile.



Note :

- T: Tamshui River Zone .
- H: Hsintien River Zone .
- K: Keelung River Zone .
- : 1~18 Test Piles .

Figure 4.3 Location of Test Piles

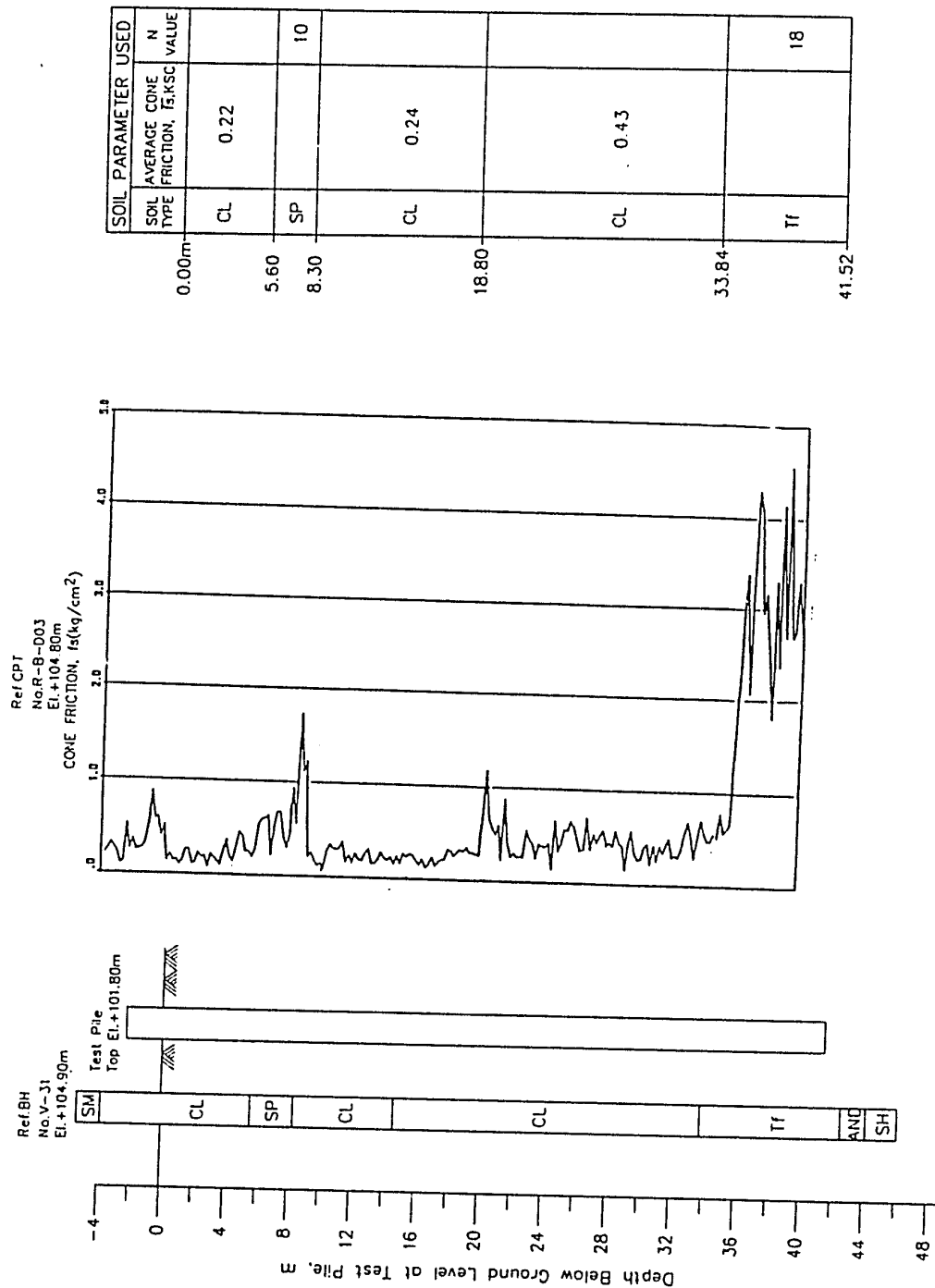


Figure 4.4 Soil Condition at Test Pile No. 1

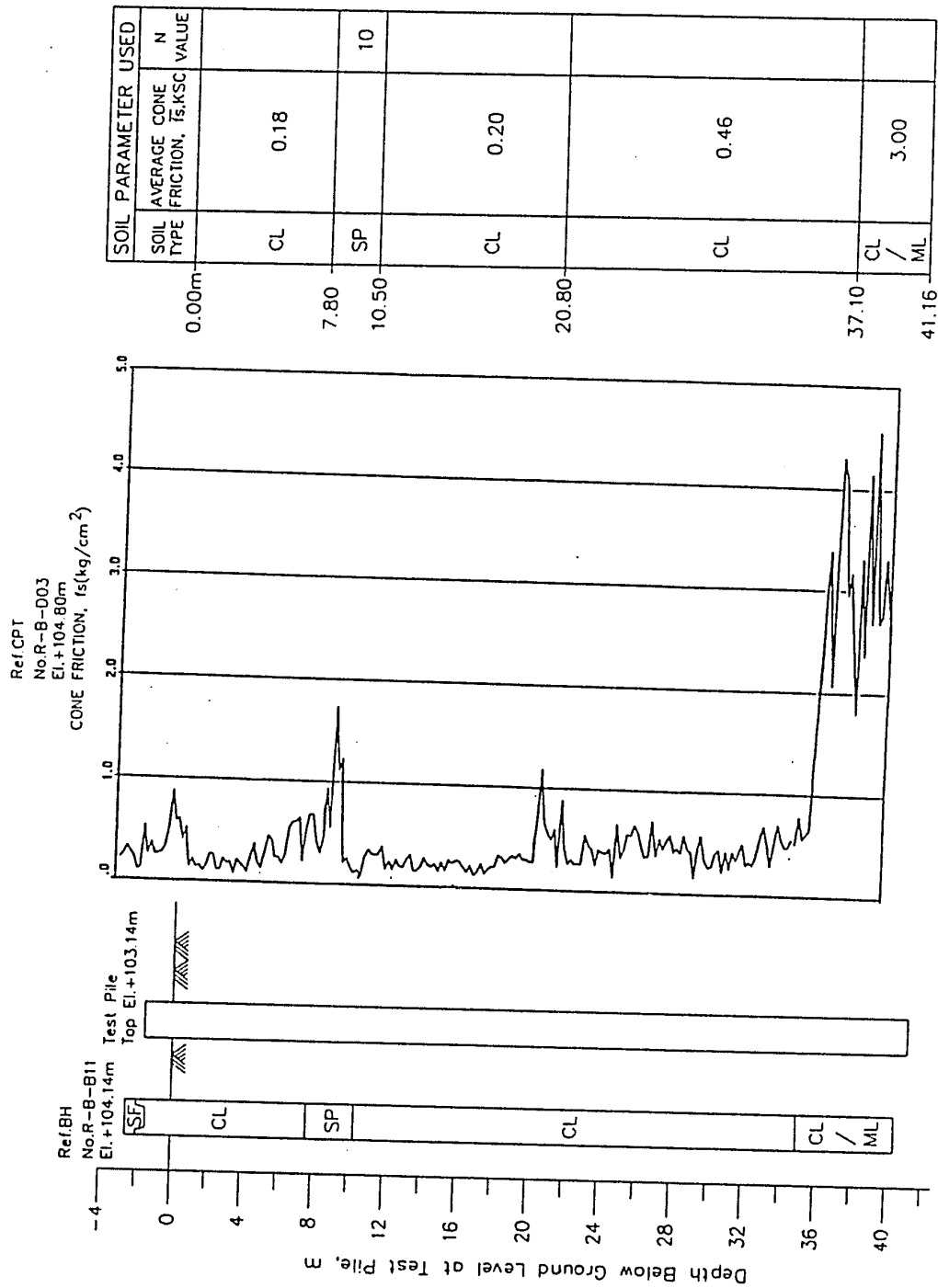


Figure 4.5 Soil Condition at Test Pile No. 2

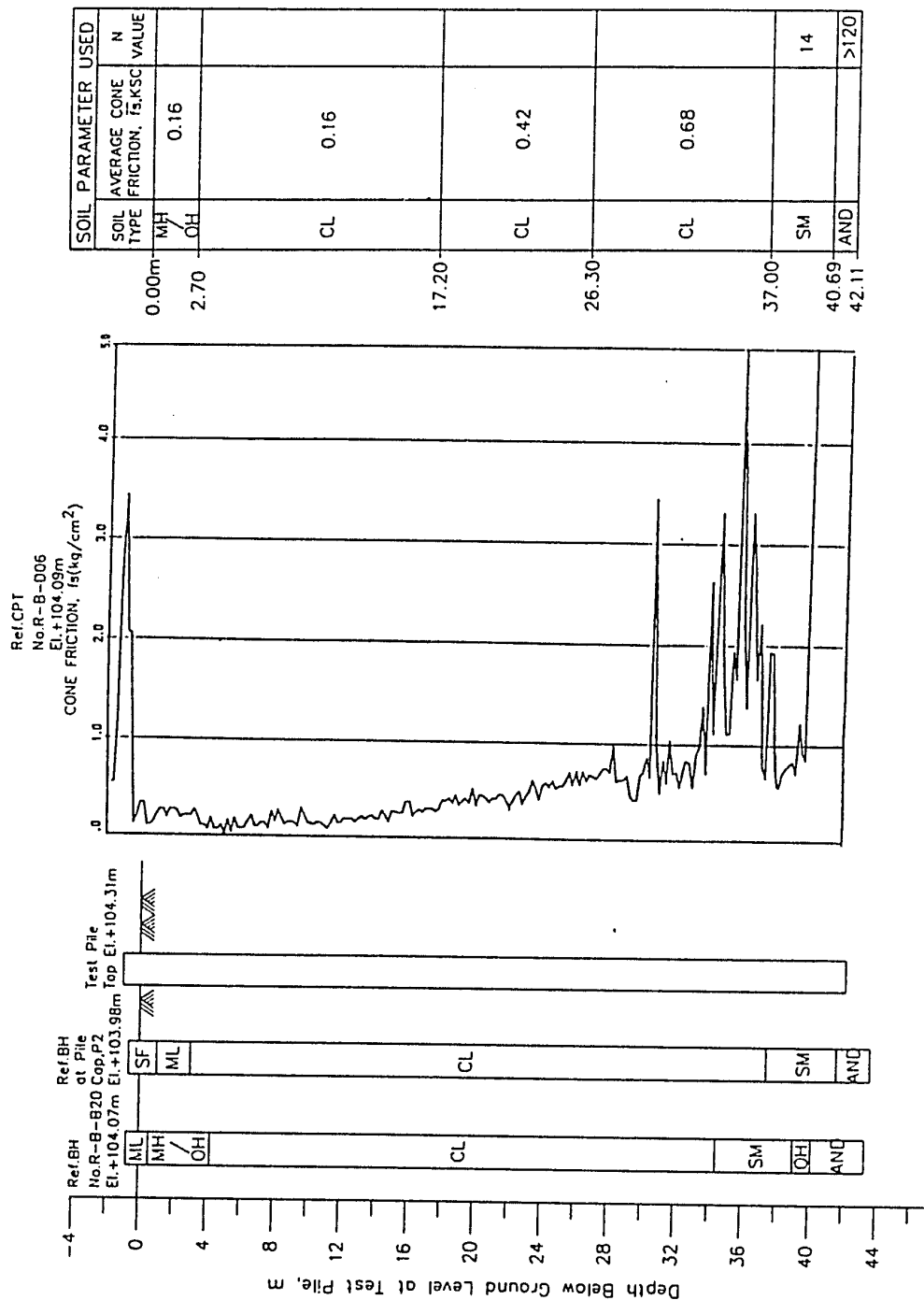


Figure 4.6 Soil Condition at Test Pile No. 3

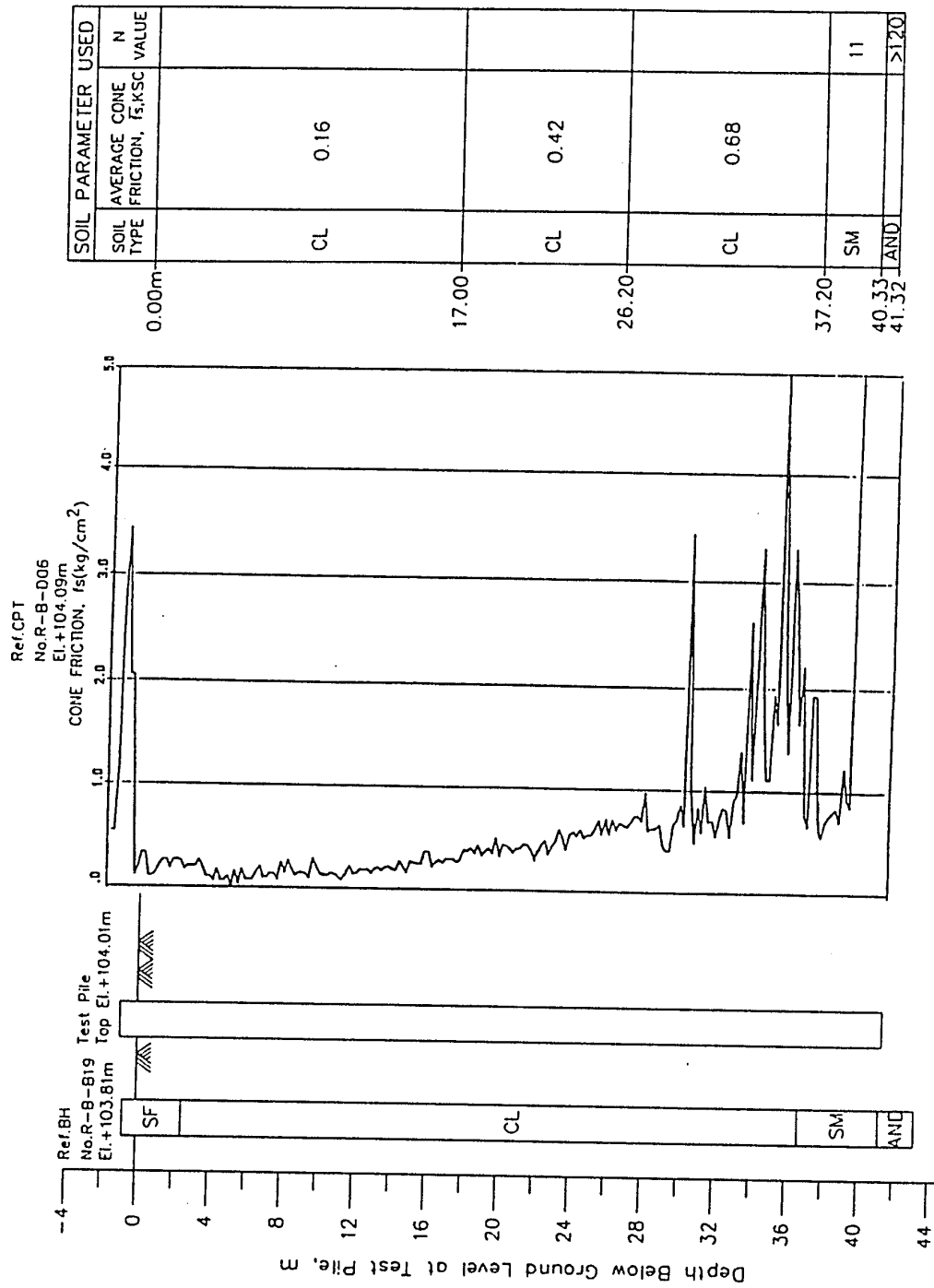


Figure 4.7 Soil Condition at Test Pile No. 4

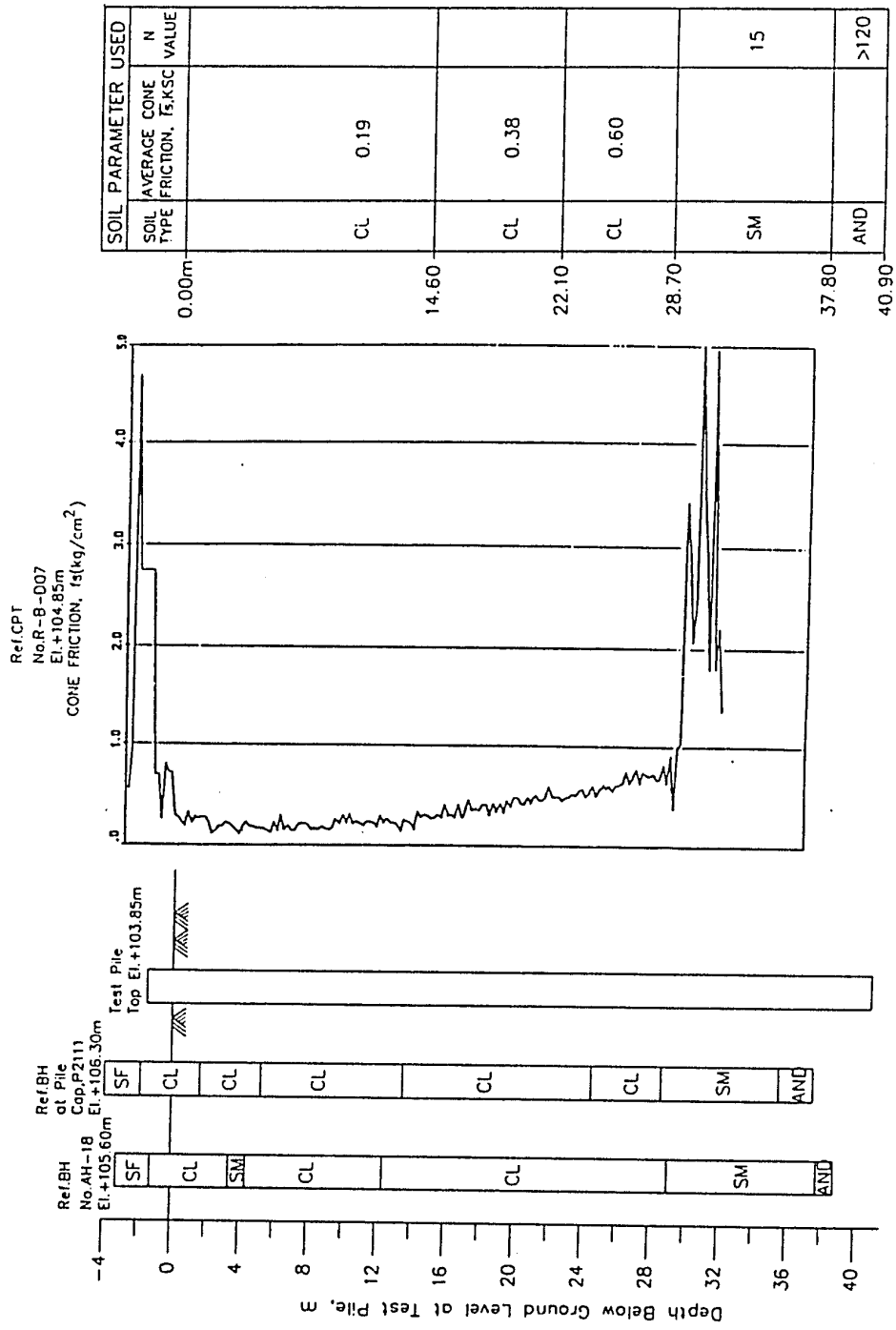


Figure 4.8 Soil Condition at Test Pile No. 5

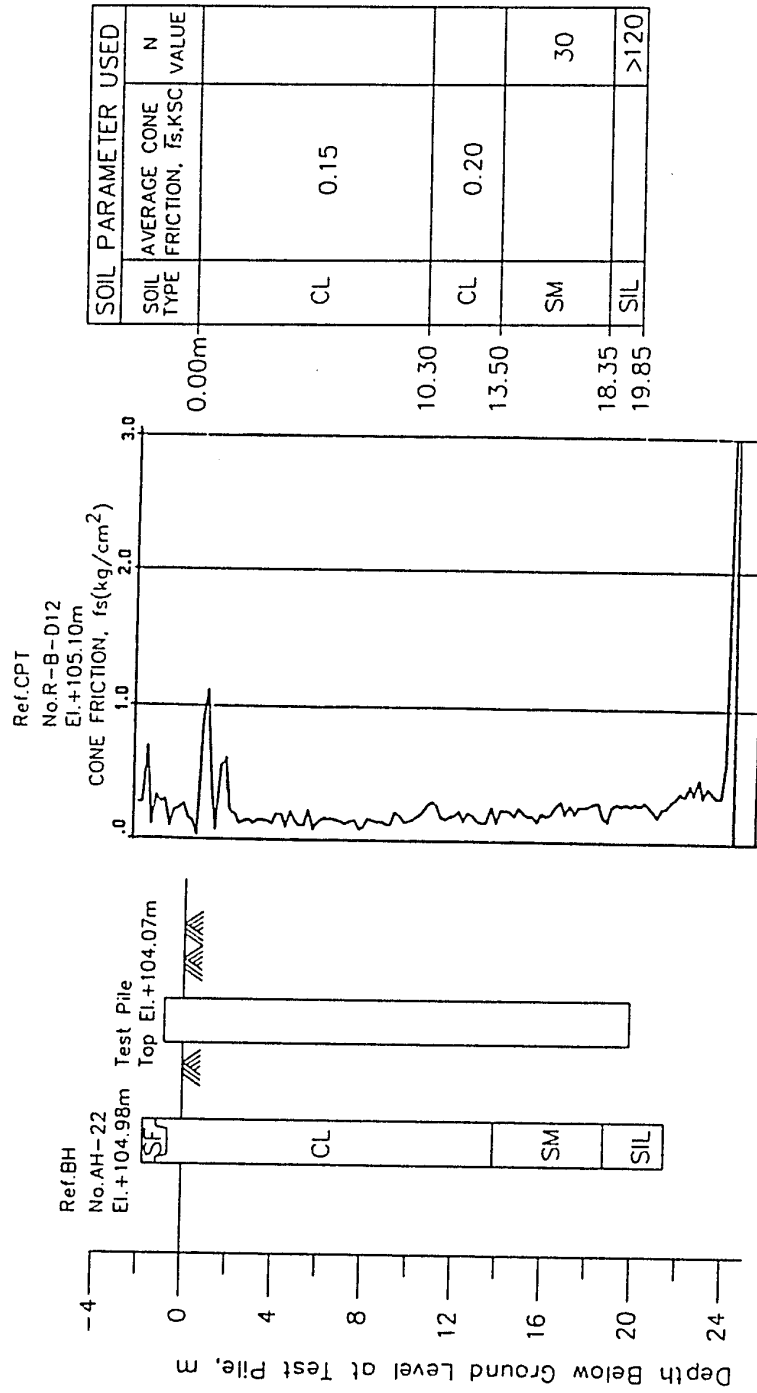


Figure 4.9 Soil Condition at Test Pile No. 6

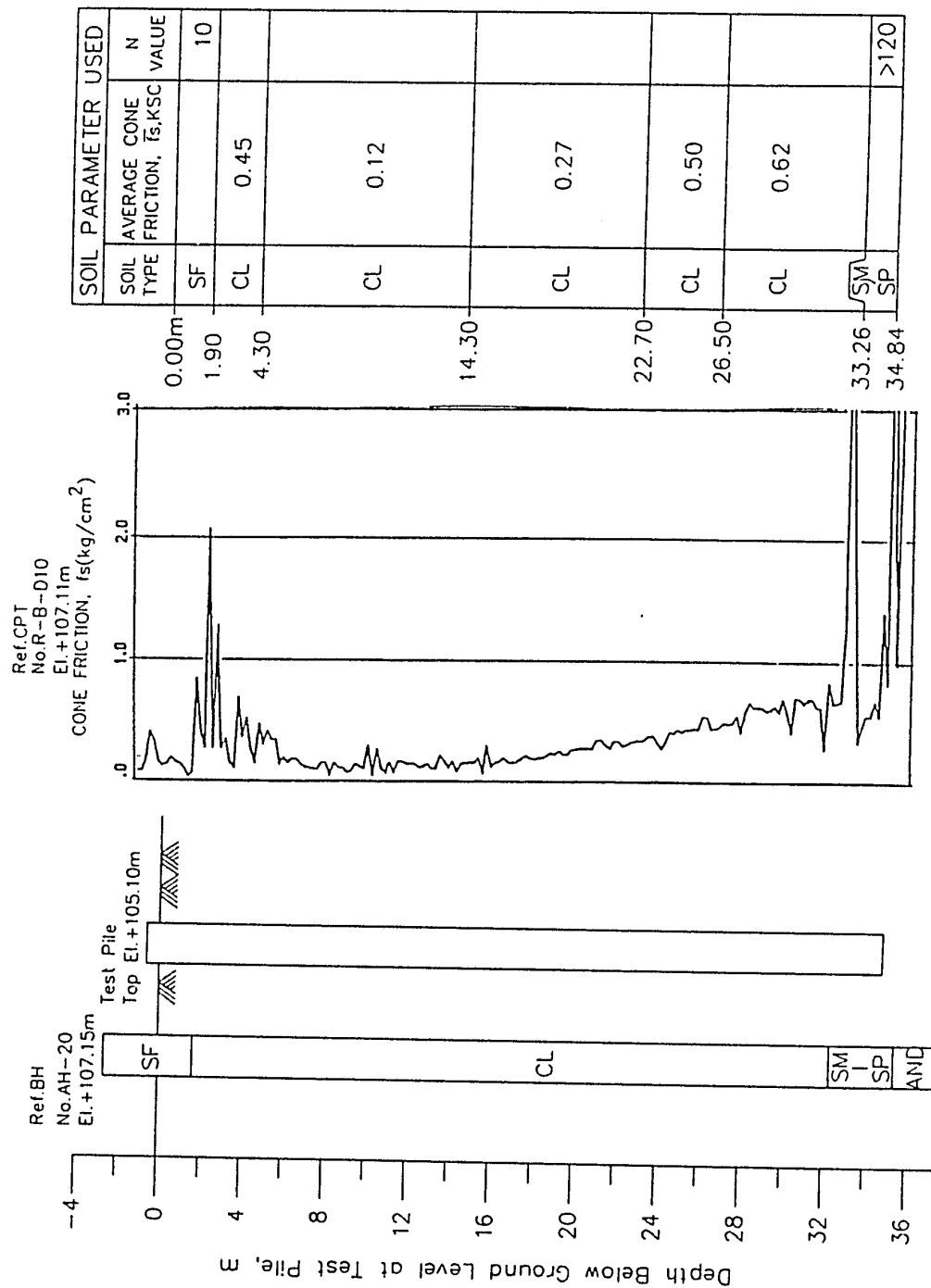


Figure 4.10 Soil Condition at Test Pile No. 7

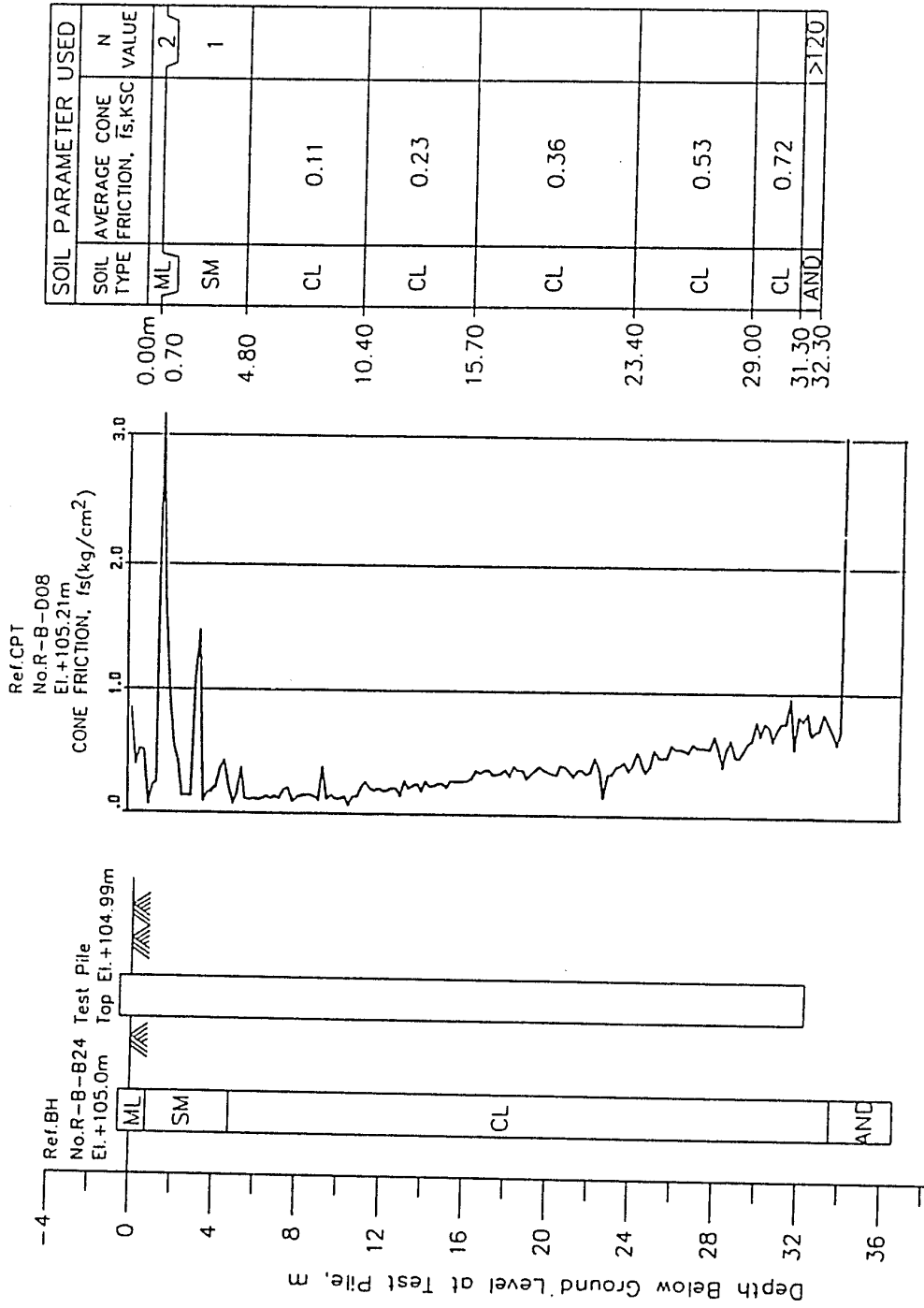


Figure 4.11 Soil Condition at Test Pile No. 8

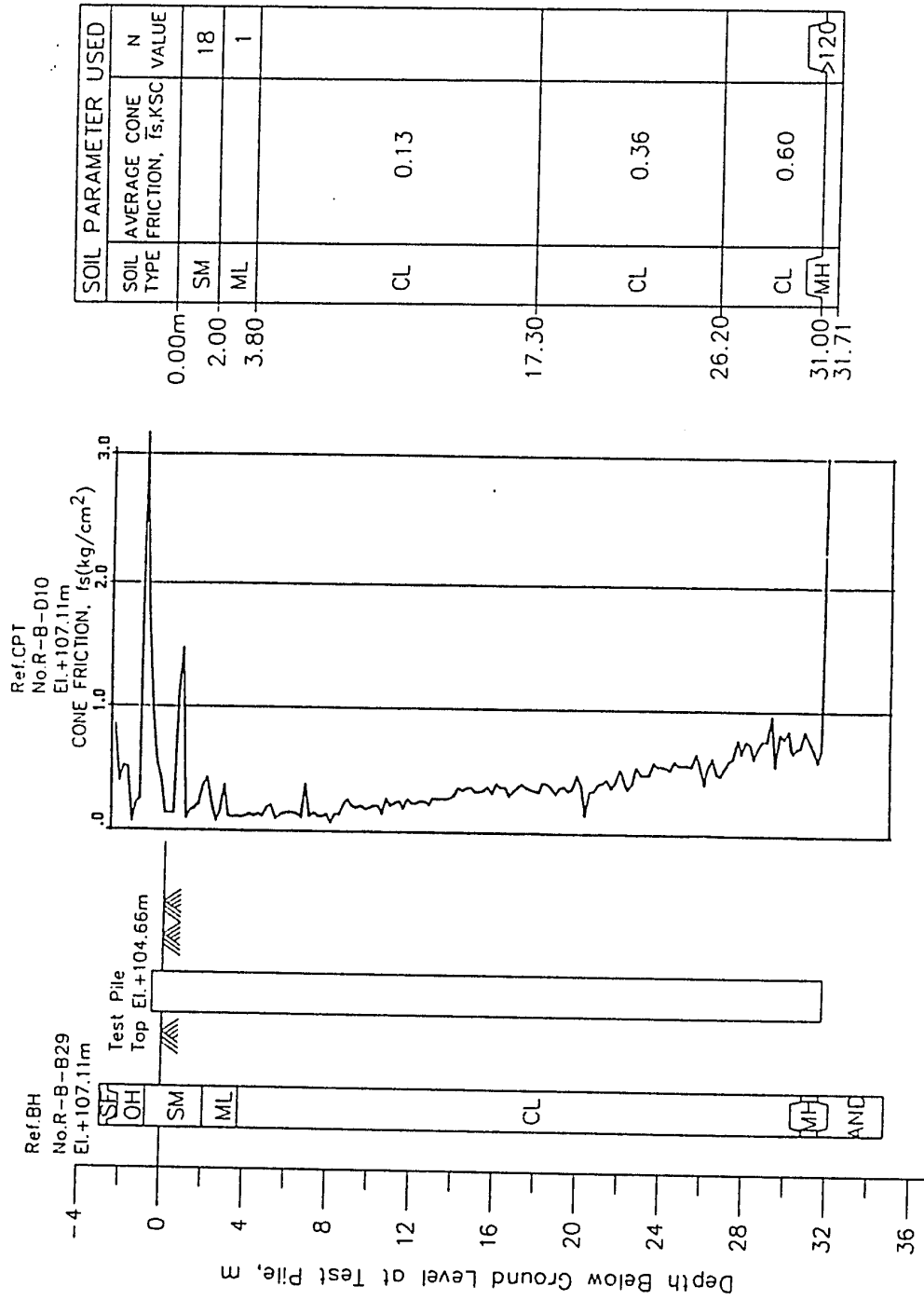


Figure 4.12 Soil Condition at Test Pile No. 9

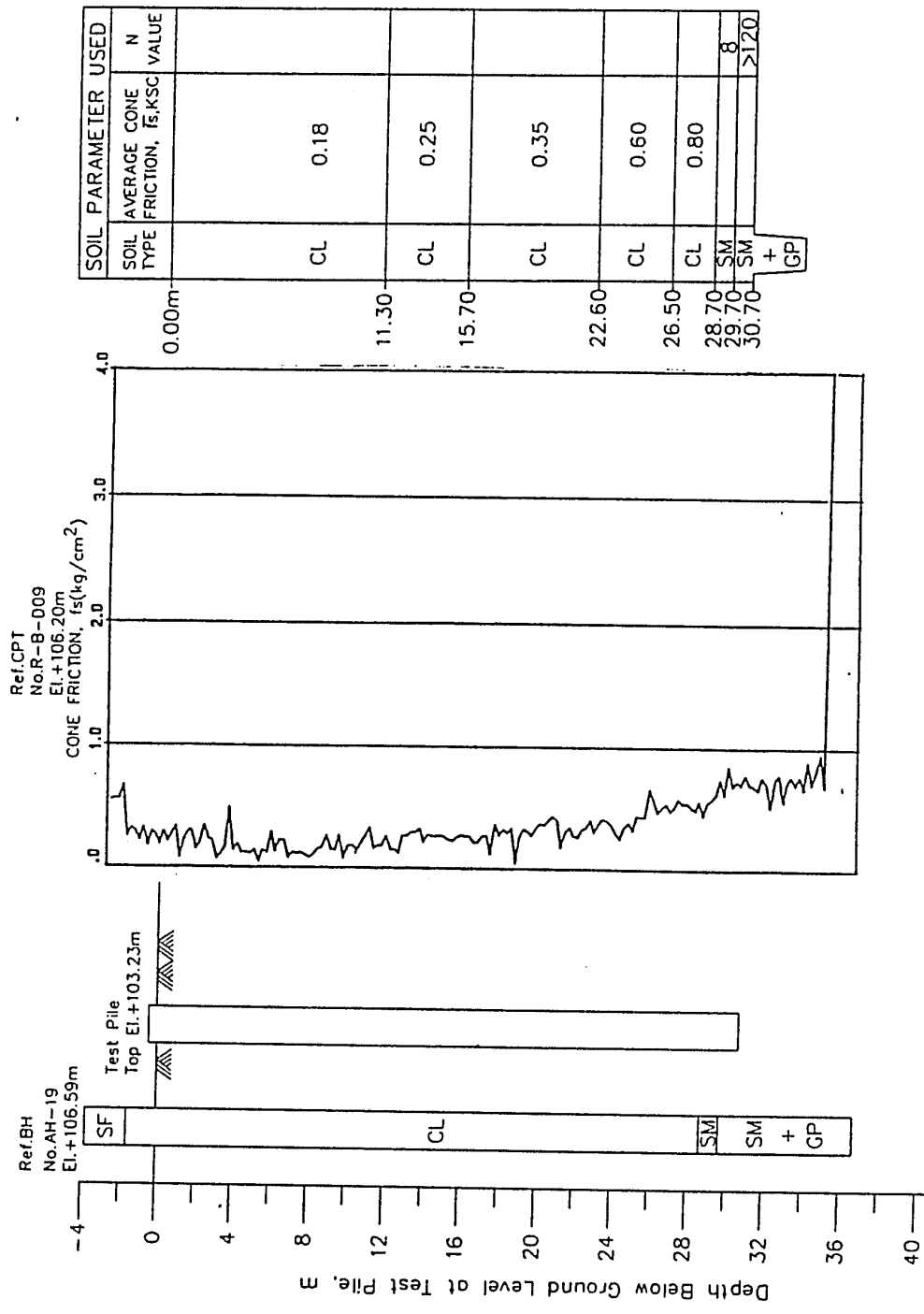


Figure 4.13 Soil Condition at Test Pile No. 10

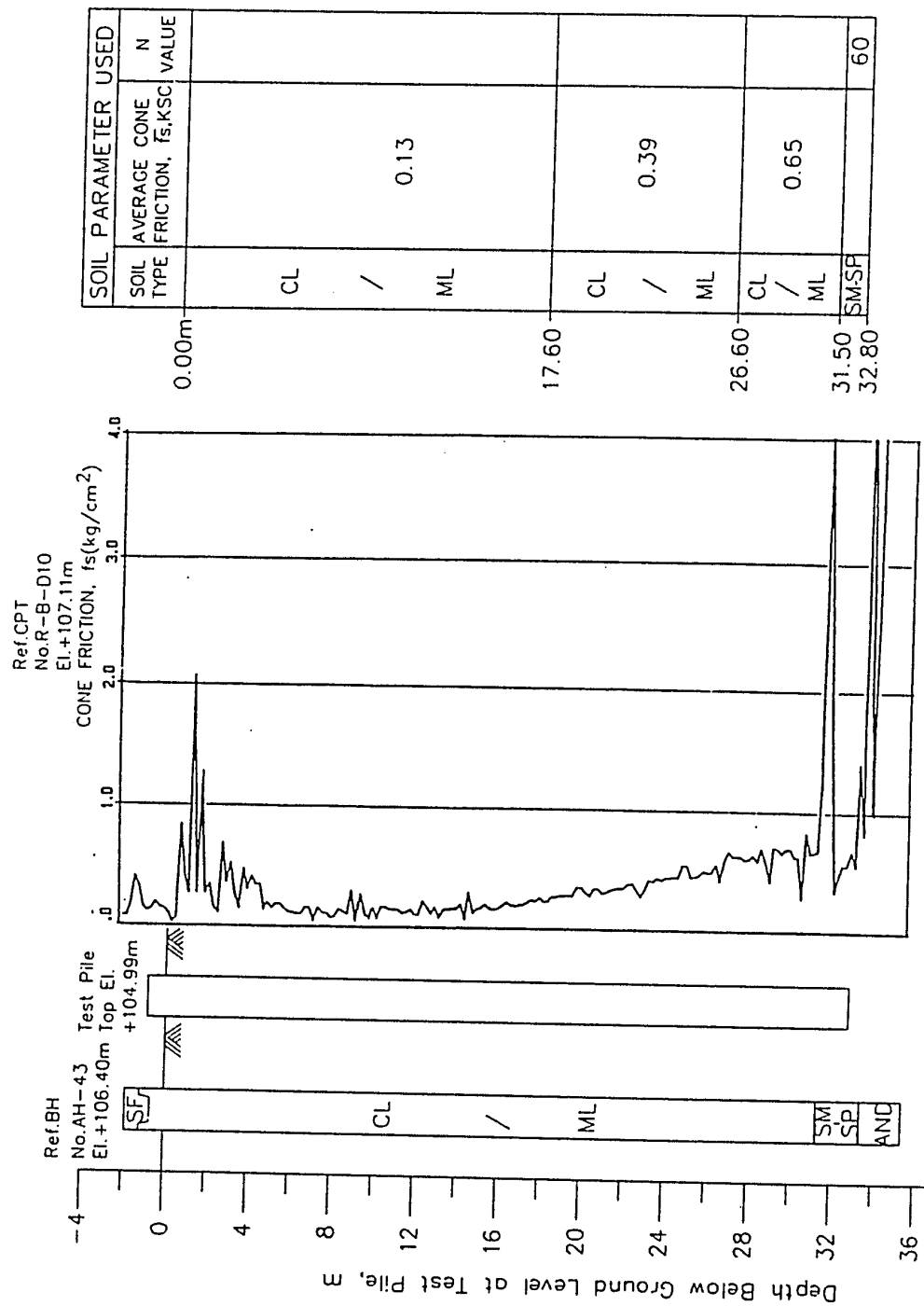


Figure 4.14 Soil Condition at Test Pile No. 11

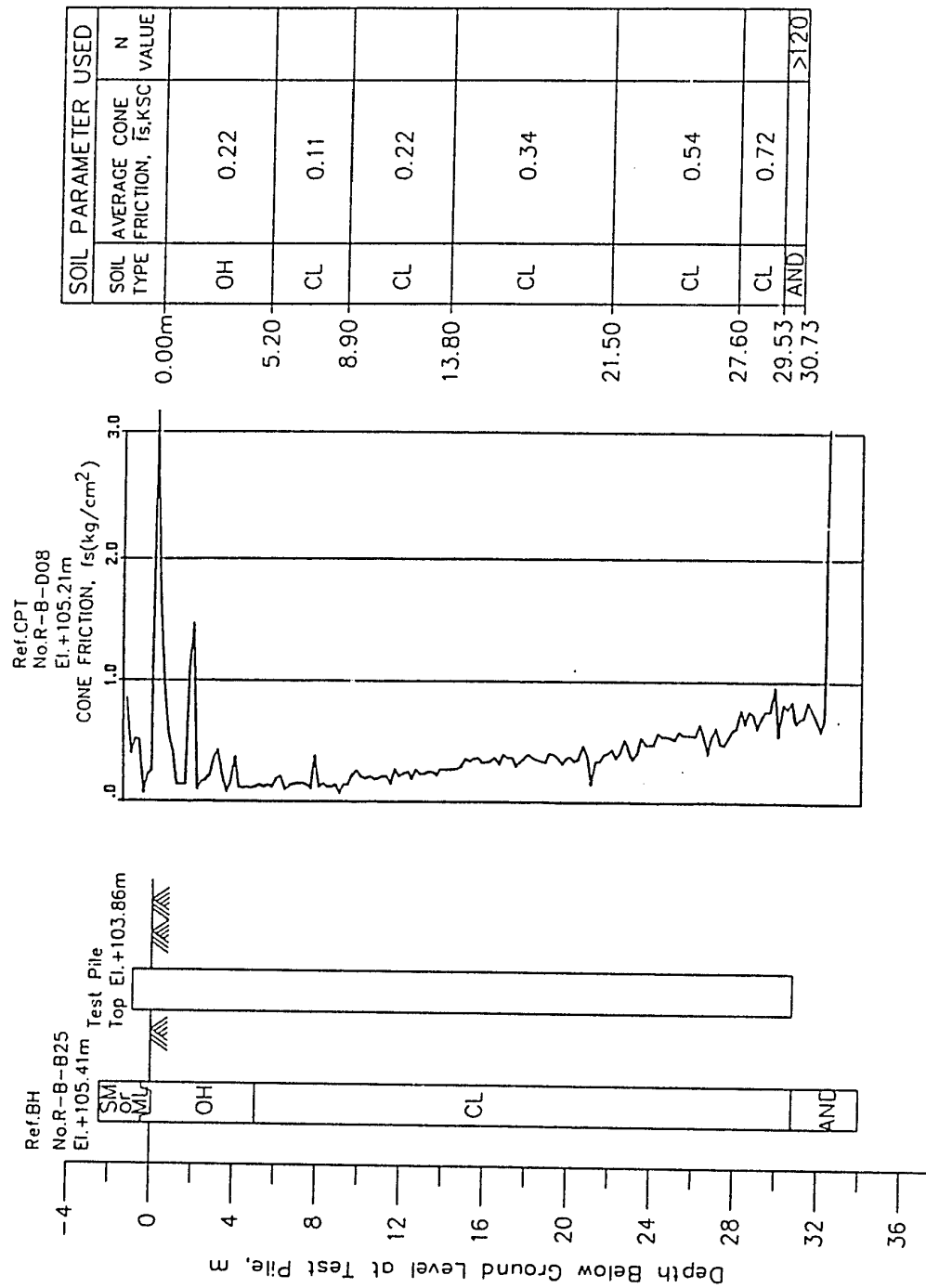


Figure 4.15 Soil Condition at Test Pile No. 12

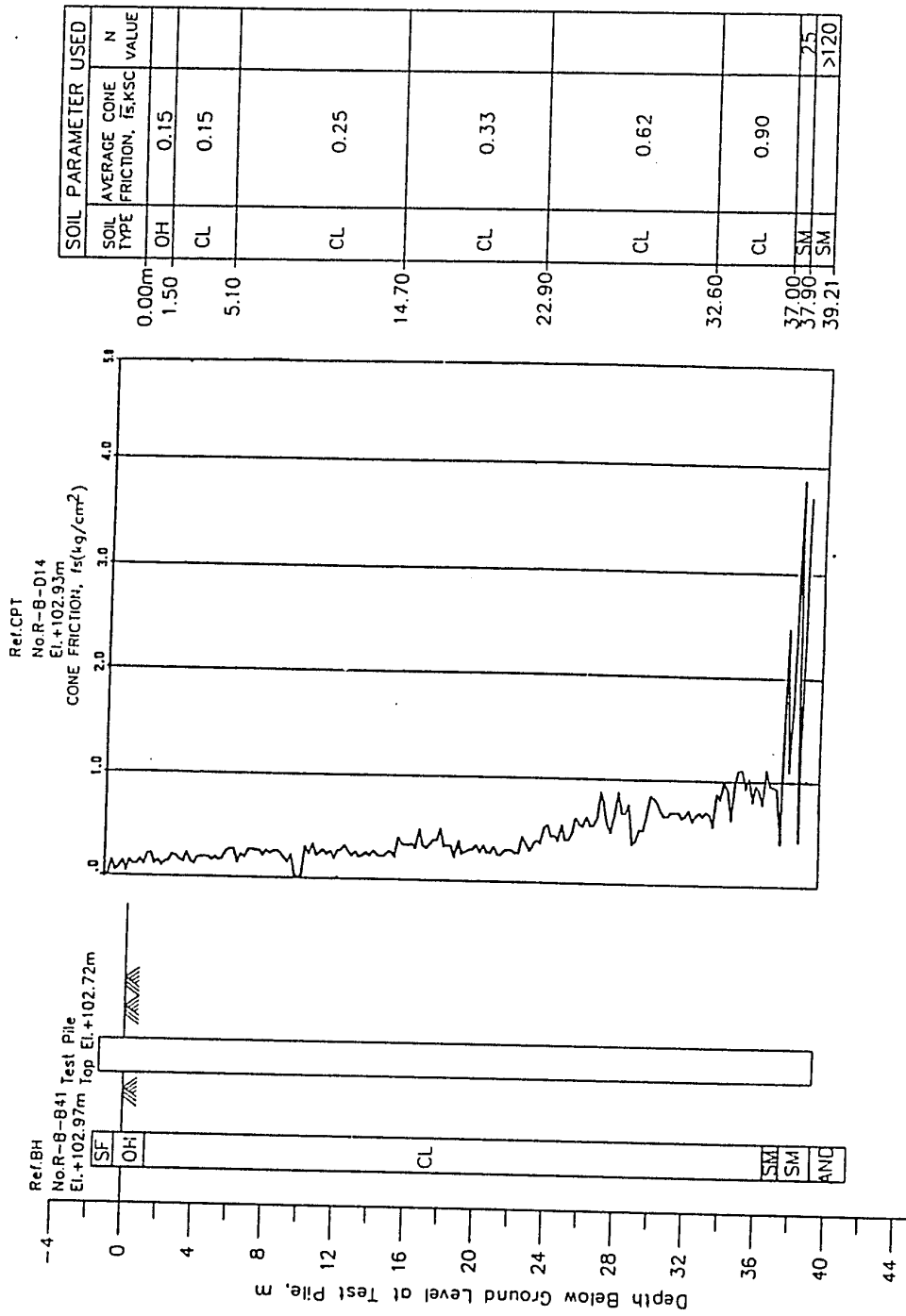


Figure 4.16 Soil Condition at Test Pile No. 13

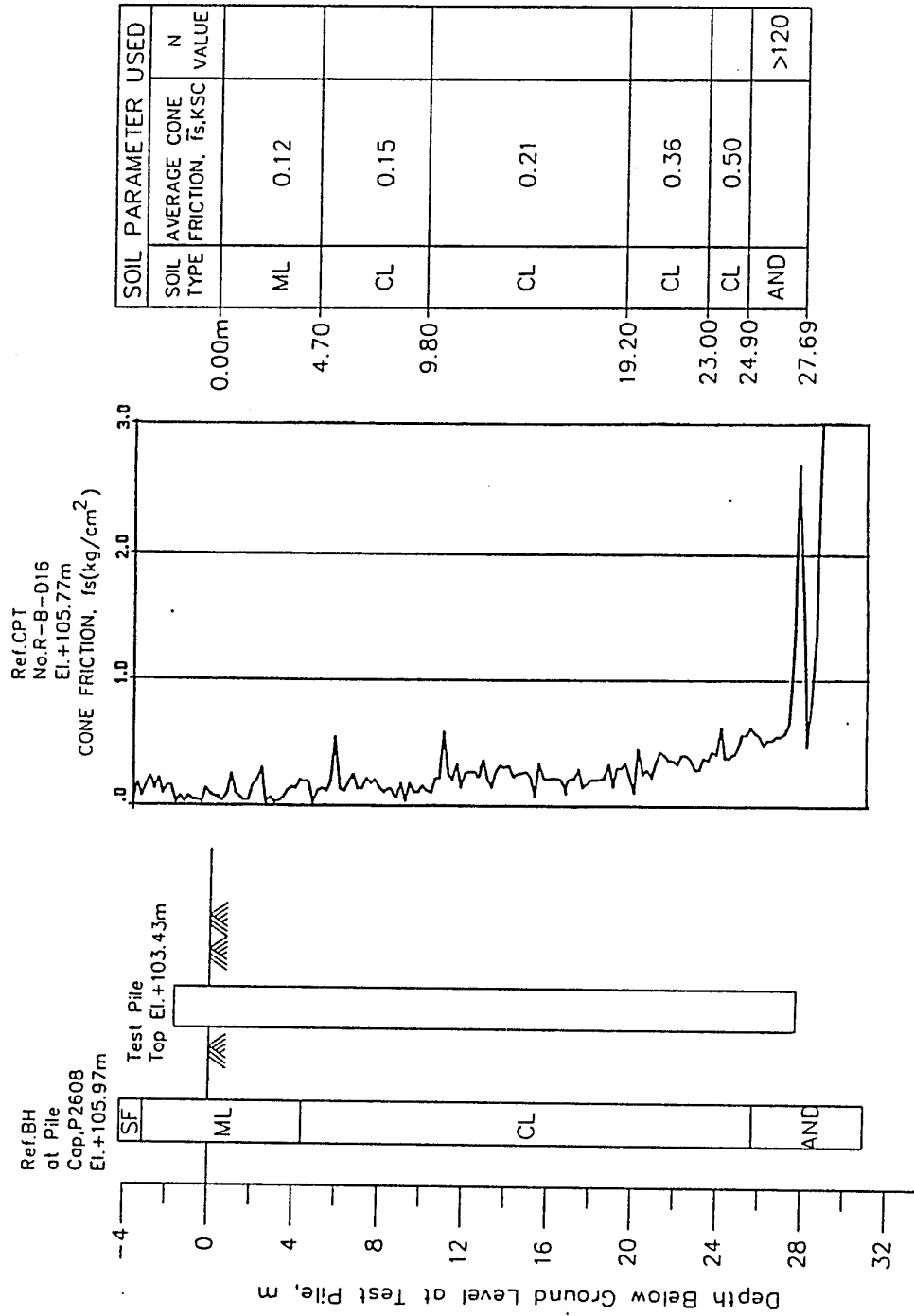


Figure 4.17 Soil Condition at Test Pile No. 14

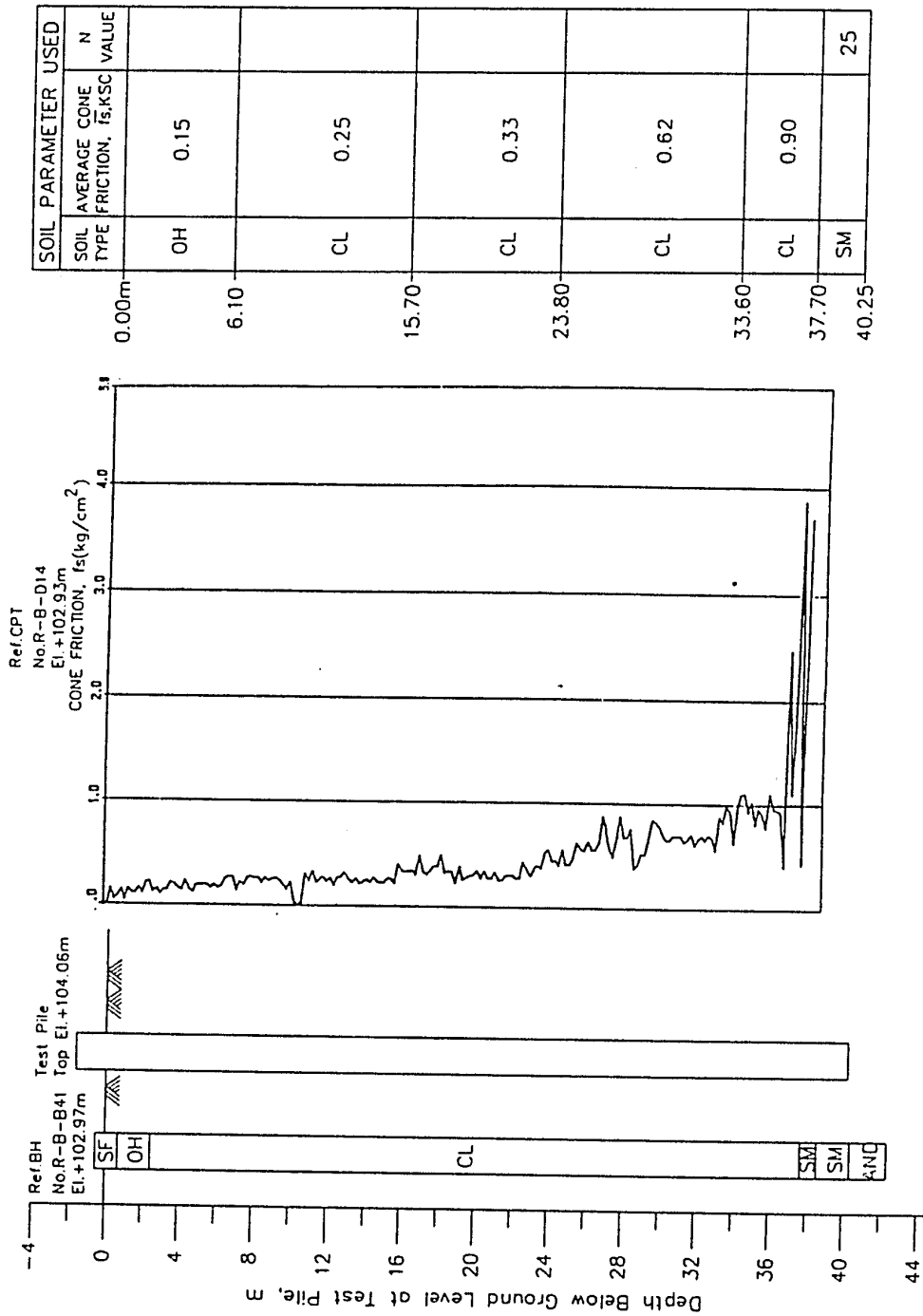


Figure 4.18 Soil Condition at Test Pile No. 15

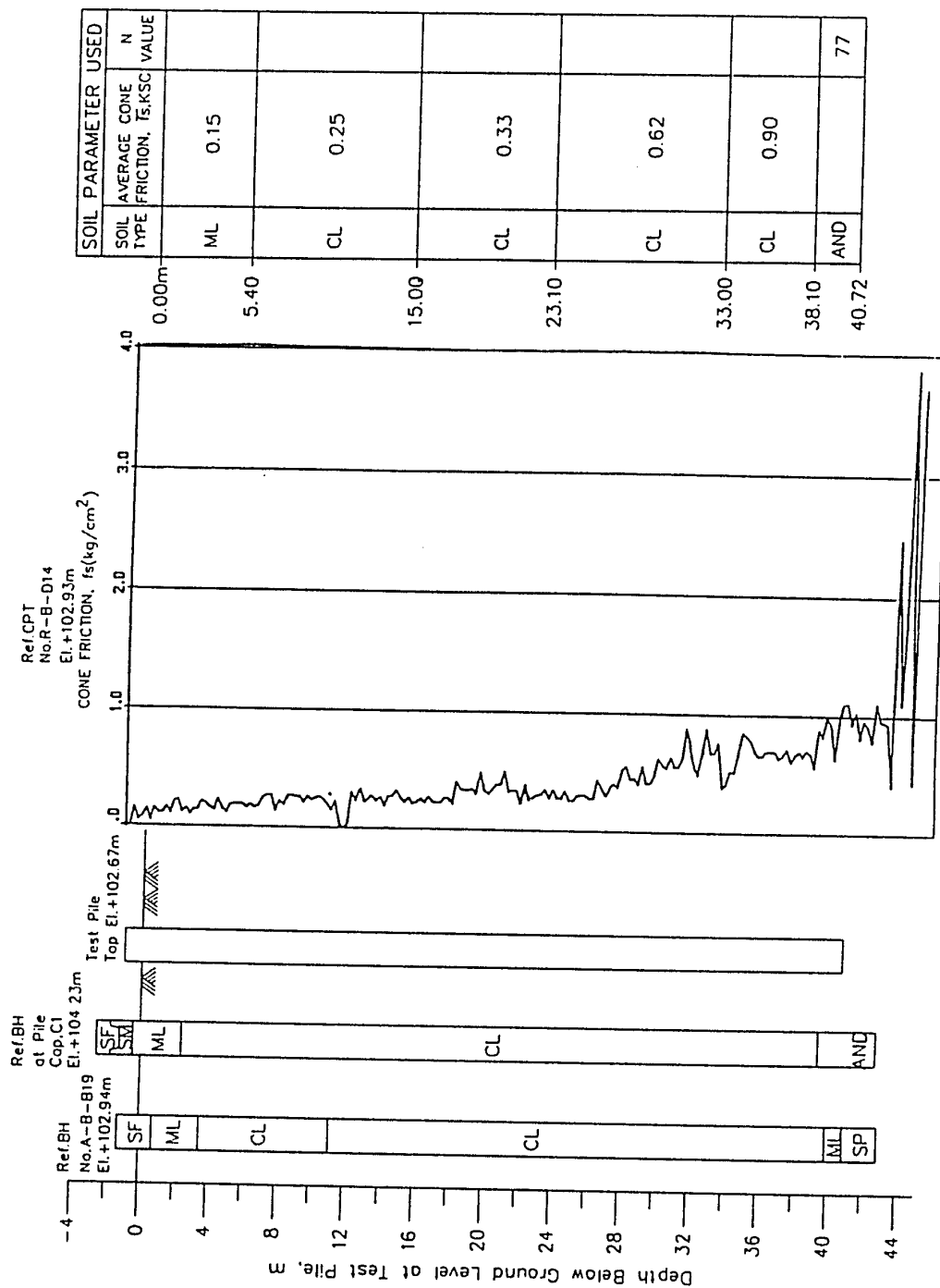


Figure 4.19 Soil Condition at Test Pile No. 16

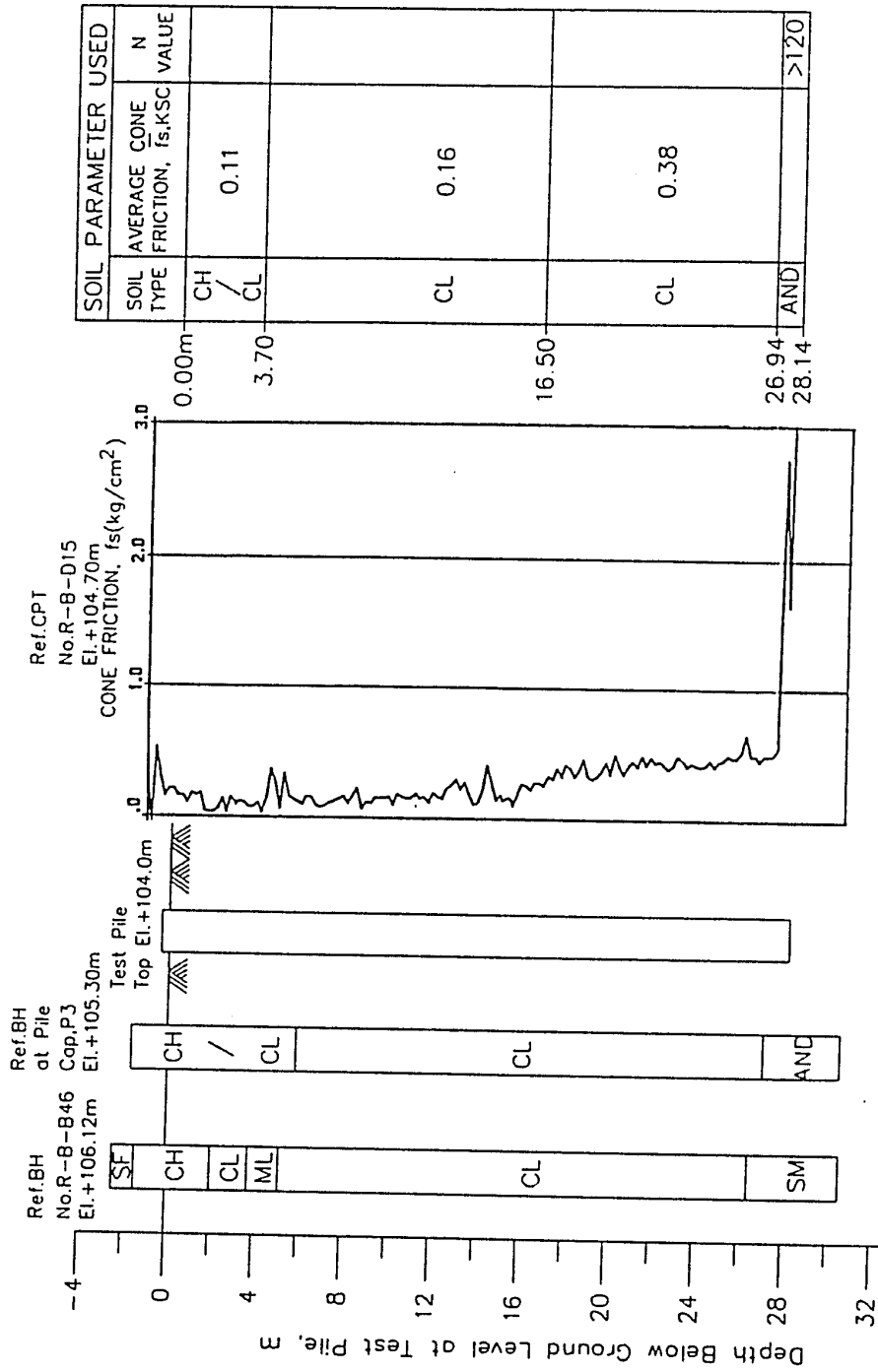


Figure 4.20 Soil Condition at Test Pile No. 17

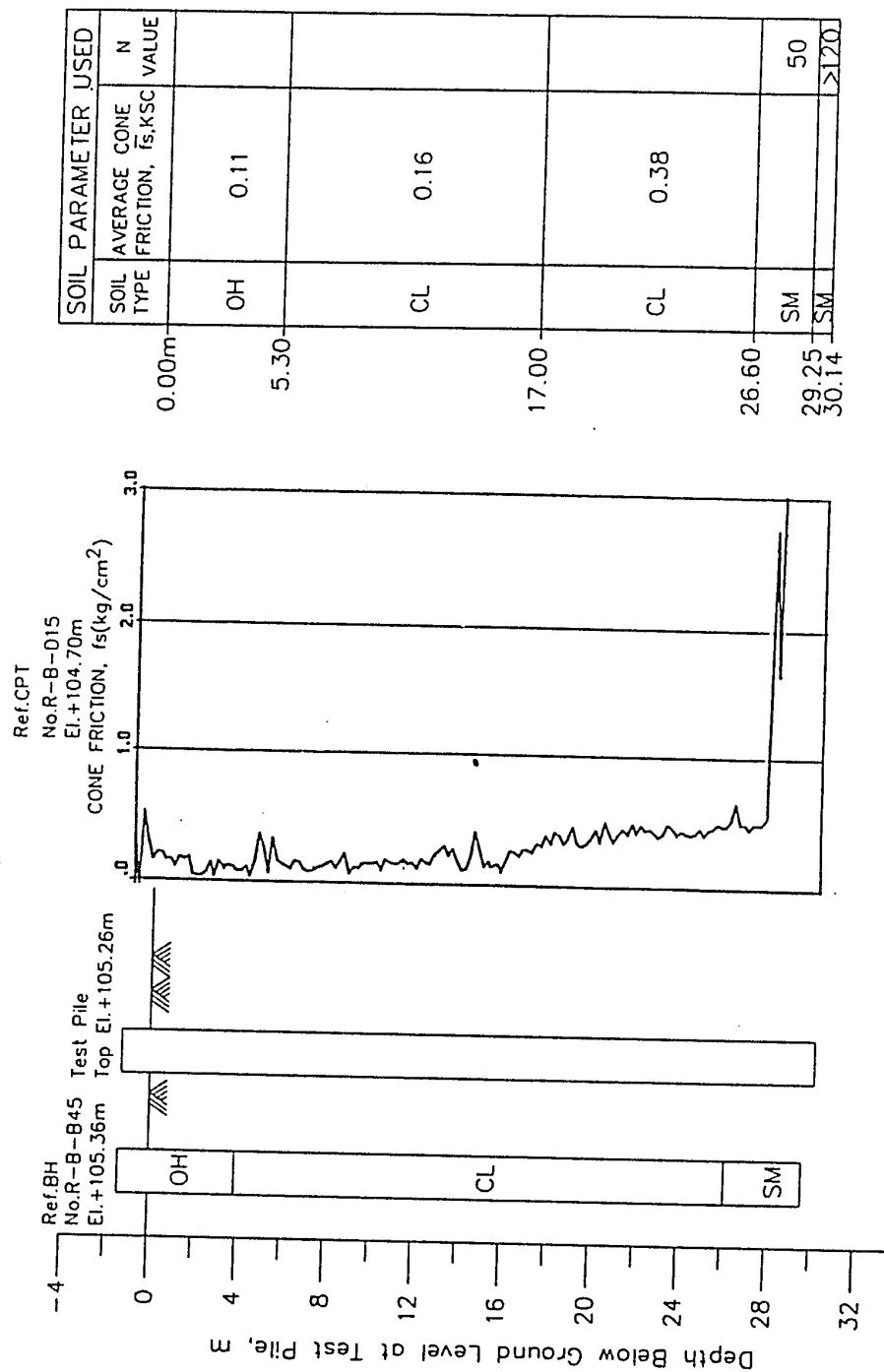


Figure 4.21 Soil Condition at Test Pile No. 18

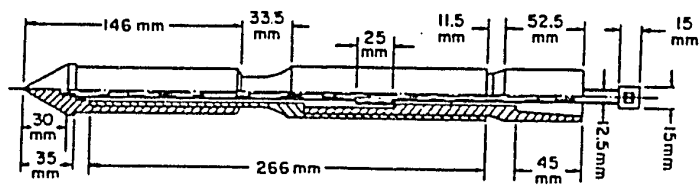
4.3 Cone penetration Test

4.3.1 Apparatus

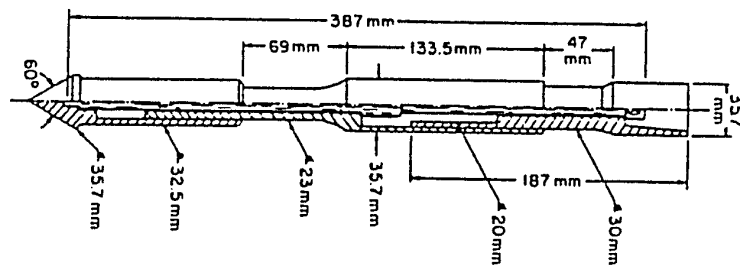
The apparatus consists of a hydraulical thrust machine (manufactured by N. V. GOUDSCHE MACHINE FABRIEK, NETHERLANDS), a set of tubes containing pressure rods inside, and cone. The mechanical friction-cone (Begemann type) as shown in Figure 4.22 was used to permit separate measurement of end bearing and side friction during the penetration. The cone has a 60-degree point angle with a projected area of 10 square centimeters.

4.3.2 Preparation for Testing

The surface of the location to be tested (approximately 2.0 m x 2.0 m area) was prepared as a level plane, the thrust machine was then set up for a thrust direction in the vertical. The thrust machine was anchored as necessary (as shown in Figure 4.23) to balance the penetration reaction during penetrating to the required depth or resistance.



Collapsed



Extended

Figure 4.22 Mechanical Friction-cone Penetrometer Tip
(Begemann Type)

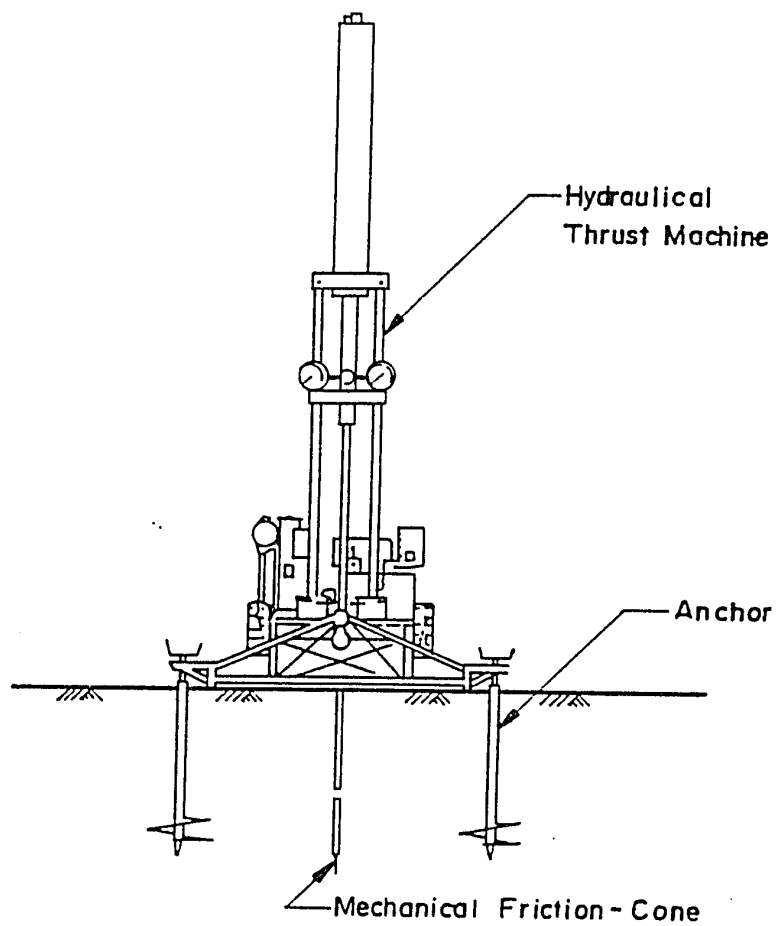


Figure 4.23 Dutch Cone Penetrometer

4.3.3 Testing procedure

The procedure was conducted in accordance with ASTM D3441 as follows:

1. Assemble together the pressure rod, tube and cone in such a way as to make contact with the hydraulic measuring device.
2. Set the tip of cone to the testing point.
3. Advance the cone downward approximately 3.5 cm by applying force with a rate of 2 cm/sec. through the pressure rod. The cone would be extended beyond the friction sleeve upon which the local side, friction resistance developed. With the penetration above, two readings could be recorded from the measuring device, namely, 1) end bearing resistance or cone resistance, 2) the sum of sleeve frictional resistance and cone resistance.
4. Repeat steps 2 and 3 from the surface to the required depth (or up to the maximum resistance that the machine could balance) with 20 cm intervals of penetration. Readings were taken at each penetration for cone resistance and sleeve frictional resistance calculations.

4.4 Pile Description and Installation

Cast-in-situ bored piles, which are called reverse circulation piles in Taiwan, have been successfully used for supporting many bridges, flyovers and high-rise buildings. For installation of this type of pile, a hole is first bored by a drill bit. To prevent collapse of the hole, the hole is always filled with clay slurry. The slurry is also used to carry away the soil cuttings by suction through the bit. New slurry is pumped into the hole to maintain a constant head of fluid. After the desired depth is reached, a reinforcement is lowered and tremie concrete is placed to completely replace the slurry.

The general procedure for the installation of the piles was as follows:

1. Drive a temporary casing of diameter approximately 15 cm larger than the pile diameter, to the depth of 3 to 5 m.
2. Start boring by a drilling bit within the casing while clay slurry was filled into the hole and the fluid level was maintained at a level not less than 1.0 m above the level of the external ground water, and generally up to the ground level (ground water was generally at 2.0 m below ground level).
3. Continue drilling to the required elevation under mud slurry, with soil cuttings carried away by suction through the bit.

4. Clean the bottom of the hole by maintaining at least 30 min. of suction through the bit.
5. Withdraw the drilling bit and lower the reinforcement.
6. Place concrete through a tremie pipe with a rubber plunger in the pipe to separate the concrete from the slurry, while keeping the tremie tip well embedded in the concrete.
7. As slurry is completely replaced by the concrete, waste the first portion of the concrete flow, as this concrete is usually contaminated with mud.
8. Extract the temporary casing.

4.5 Test Loading Procedure and Results

The procedure for conducting the load tests was generally in accordance with ASTM Standard Method of Testing Piles Under Static Axial Compressive Load (D1143-81). All of the devices for measuring deflection at the top of the pile, including four dial gages, and an engineer's level were employed as described by ASTM. The load was applied by jacking against an overhead reaction with a hydraulic jack, which was monitored by an accurately calibrated hydraulic gage. The reaction for each test pile was provided by four, same diameter, tension piles. The distance between test pile and tension piles was at least

| Loading Stage | Applied Load, ton | Maximum Duration, hr. | Pile Butt Settlement, mm |
|---------------|-------------------|-----------------------|--------------------------|
| 0 | 0.0 | 0.0 | 0.00 |
| 1 | 94.0 | 1.0 | 0.28 |
| 2 | 188.0 | 1.0 | 0.82 |
| 3 | 282.0 | 1.0 | 1.66 |
| 4 | 376.0 | 1.0 | 2.26 |
| 5 | 470.0 | 2.0 | 3.47 |
| 6 | 564.0 | 2.0 | 4.77 |
| 7 | 658.0 | 2.0 | 6.38 |
| 8 | 752.0 | 2.0 | 8.23 |
| 8 | 752.0 | 12.0 | 8.91 |
| 9 | 564.0 | 1.0 | 8.57 |
| 10 | 376.0 | 1.0 | 7.42 |
| 11 | 188.0 | 1.0 | 5.30 |
| 12 | 0.0 | 12.0 | 2.69 |
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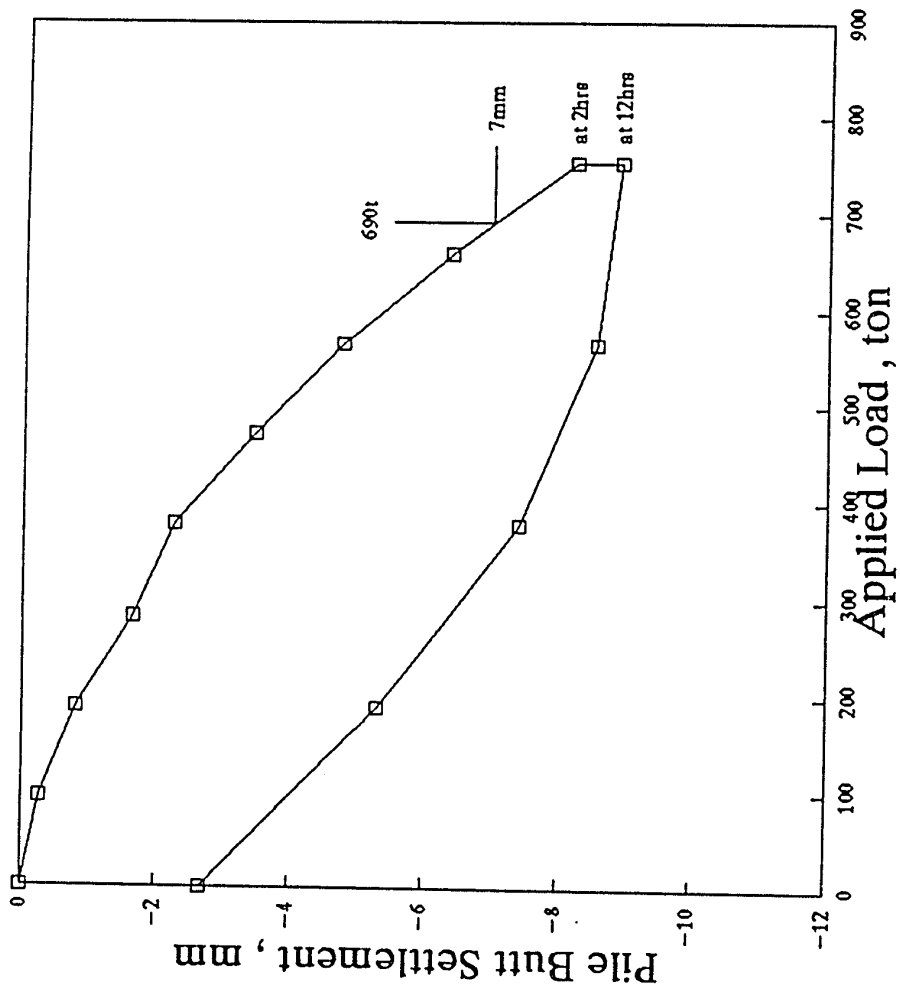


Figure 4.25 Pile Load Test Result of Test Pile No. 2

| Loading Stage | Applied Load, ton | Maximum Duration, hr. | Pile Butt Settlement, mm |
|---------------|-------------------|-----------------------|--------------------------|
| 0 | 0.0 | 0.0 | 0.00 |
| 1 | 145.8 | 1.5 | 0.49 |
| 2 | 291.5 | 1.0 | 1.19 |
| 3 | 437.3 | 1.0 | 2.16 |
| 4 | 583.0 | 1.0 | 3.20 |
| 5 | 728.8 | 2.0 | 4.51 |
| 6 | 874.5 | 1.0 | 6.02 |
| 7 | 1020.5 | 2.0 | 7.84 |
| 8 | 1166.0 | 2.0 | 11.94 |
| 9 | 0.0 | 3.0 | 4.54 |
| 10 | 1166.0 | 12.0 | 14.85 |
| 11 | 874.5 | 1.0 | 14.64 |
| 12 | 1166.0 | 13.0 | 17.53 |
| 13 | 874.5 | 1.0 | 16.64 |
| 14 | 583.0 | 1.0 | 14.47 |
| 15 | 291.5 | 1.0 | 11.80 |
| 16 | 0.0 | 12.0 | 7.98 |
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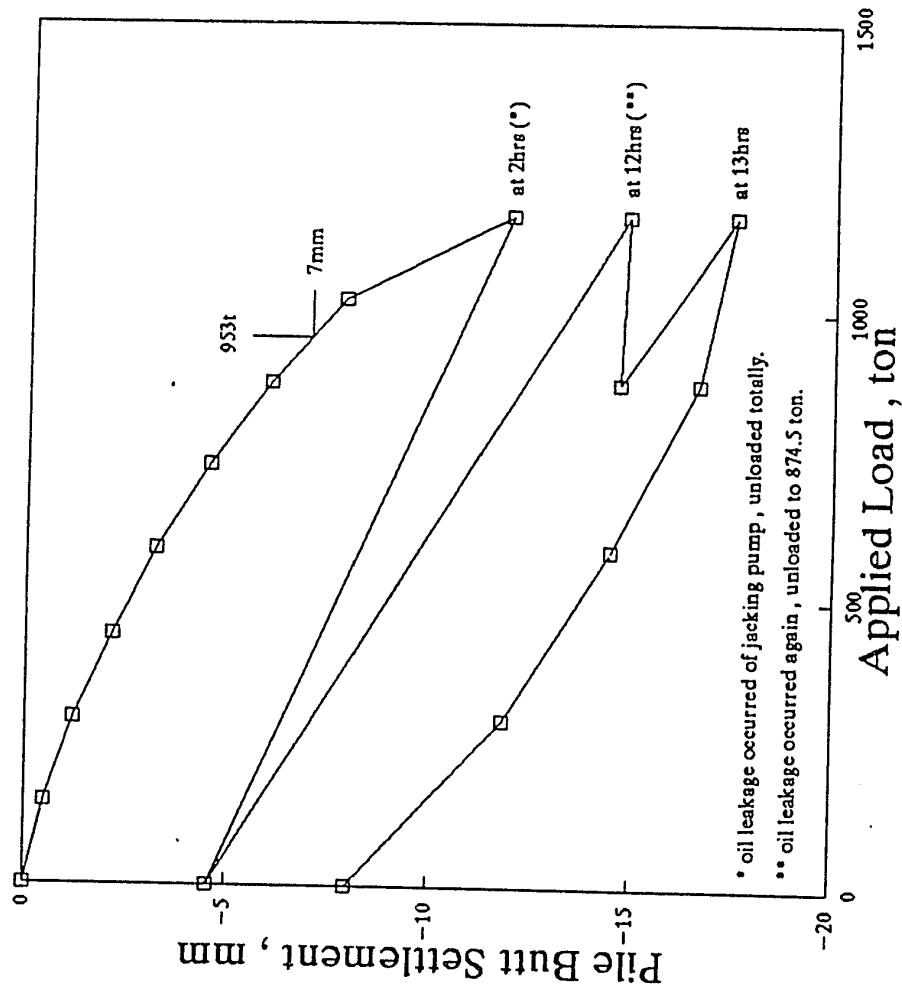
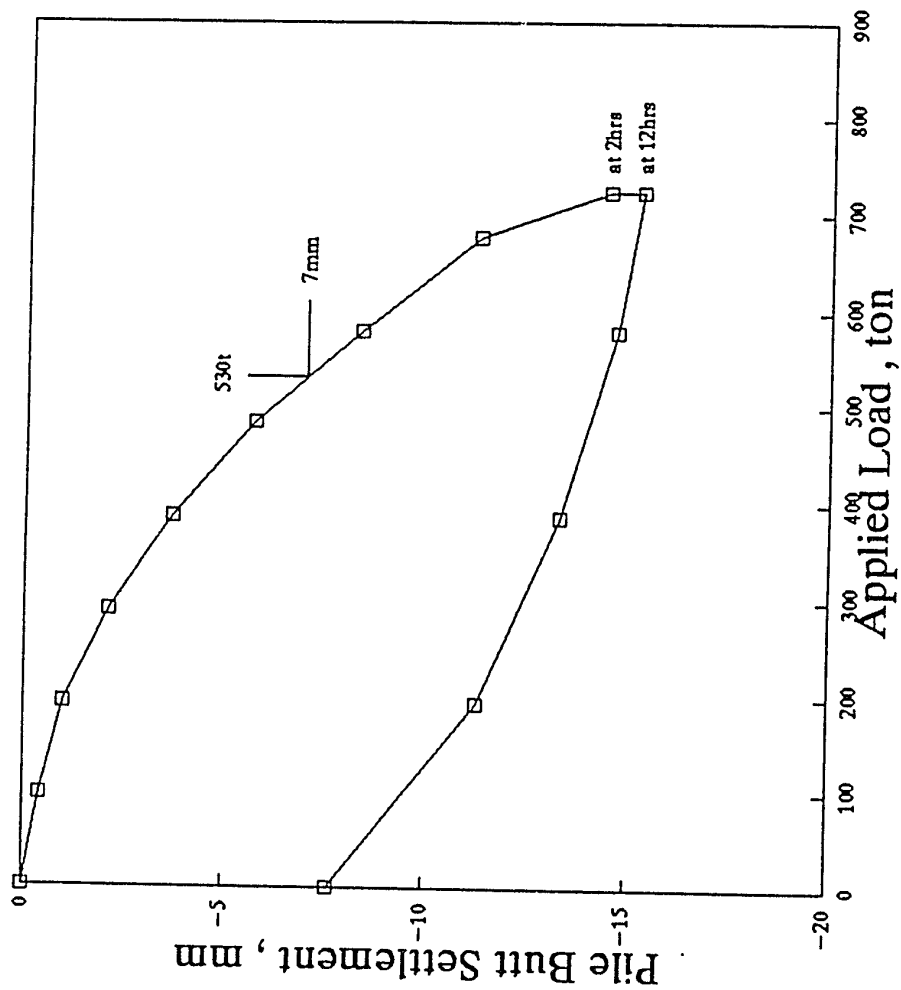


Figure 4.27 Pile Load Test Result of Test Pile No. 4



| Loading Stage | Applied Load, ton | Maximum Duration, hr. | Pile Butt Settlement, mm |
|---------------|-------------------|-----------------------|--------------------------|
| 0 | 0.0 | 0.0 | 0.00 |
| 1 | 96.5 | 1.0 | 0.44 |
| 2 | 193.0 | 1.0 | 1.01 |
| 3 | 289.5 | 1.5 | 2.13 |
| 4 | 386.0 | 1.5 | 3.68 |
| 5 | 482.5 | 1.5 | 5.73 |
| 6 | 579.0 | 2.0 | 8.33 |
| 7 | 675.5 | 2.0 | 11.27 |
| 8 | 722.0 | 2.0 | 14.46 |
| 8 | 722.0 | 12.0 | 15.28 |
| 9 | 579.0 | 1.0 | 14.68 |
| 10 | 386.0 | 1.0 | 13.30 |
| 11 | 193.0 | 1.0 | 11.30 |
| 12 | 0.0 | 12.0 | 7.63 |
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Figure 4.29 Pile Load Test Result of Test Pile No. 6

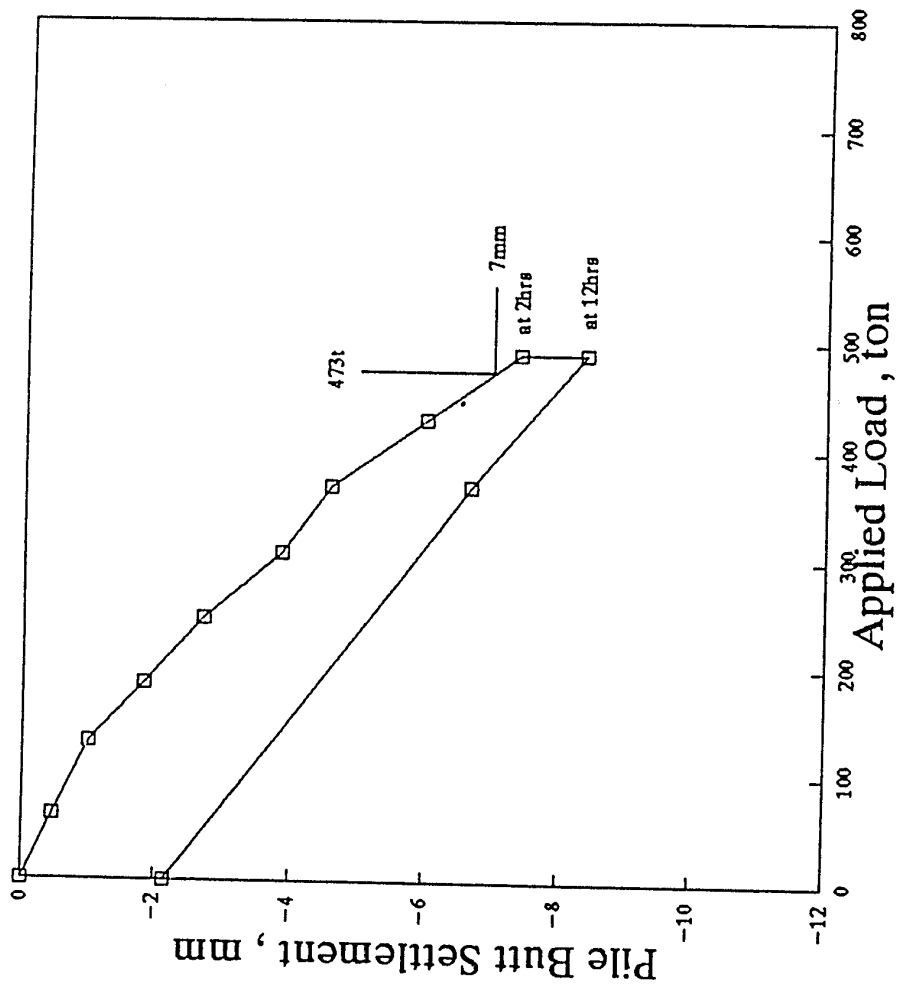
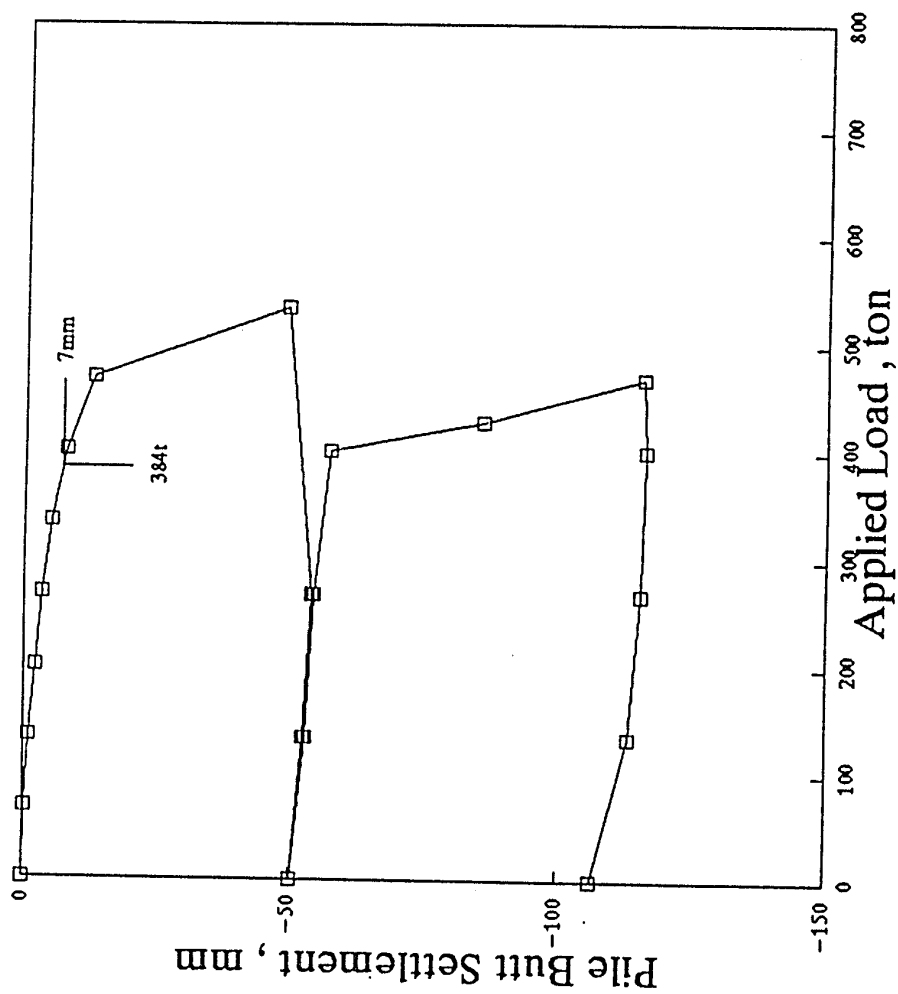
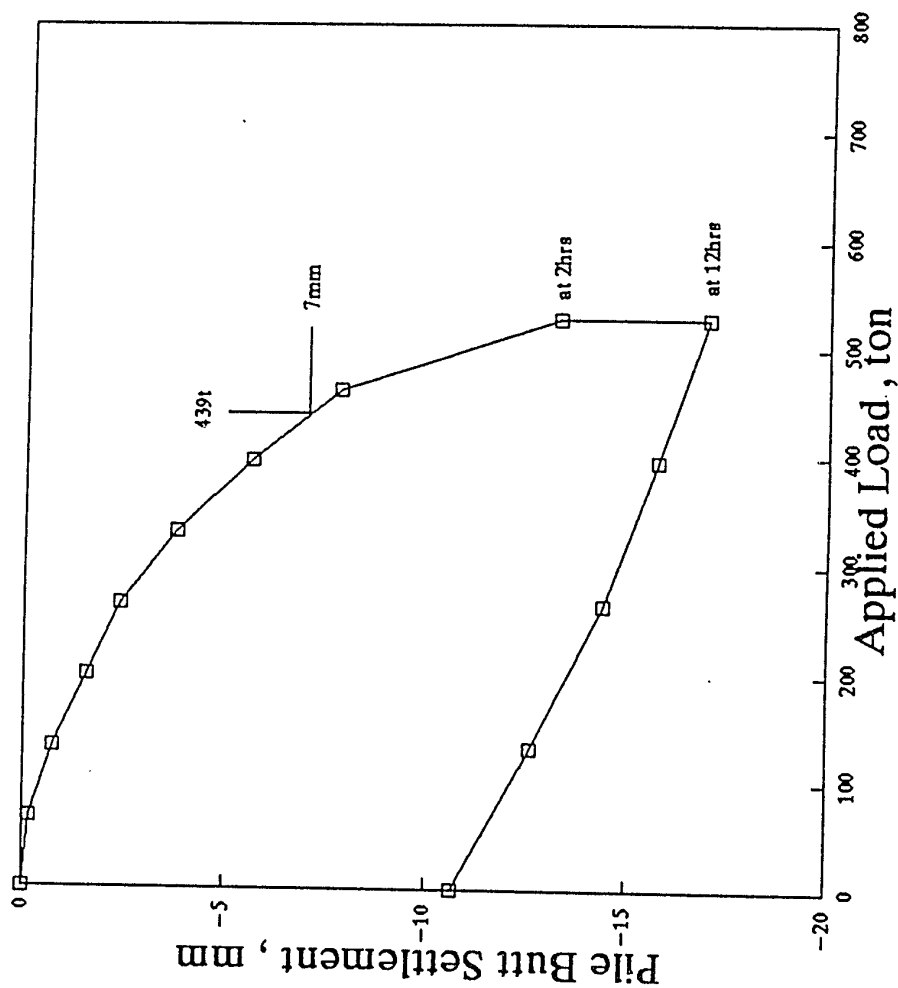


Figure 4.30 Pile Load Test Result of Test Pile No. 7



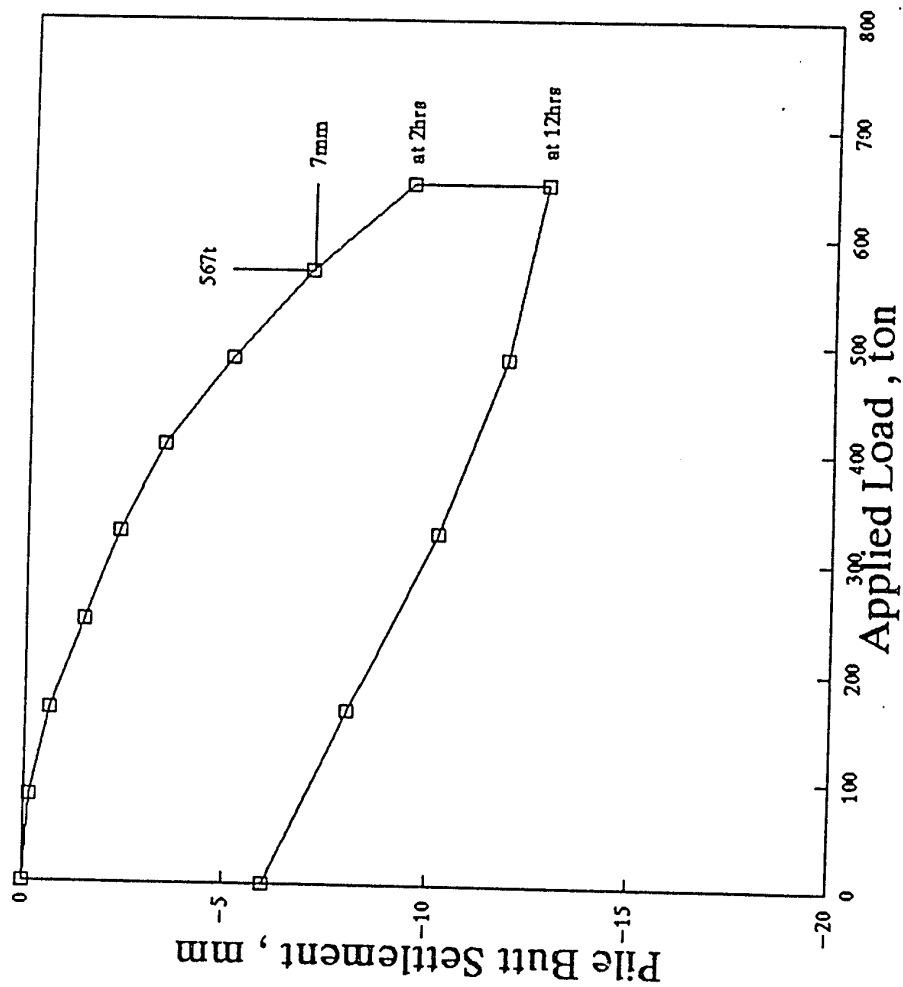
| Loading Stage | Applied Load, ton | Maximum Duration, hr. | Pile Butt Settlement, mm |
|---------------|-------------------|-----------------------|--------------------------|
| 0 | 0.0 | 0.00 | 0.00 |
| 1 | 66.9 | 1.00 | 0.39 |
| 2 | 133.8 | 2.00 | 1.17 |
| 3 | 200.7 | 2.00 | 2.20 |
| 4 | 267.6 | 2.00 | 3.54 |
| 5 | 334.5 | 2.00 | 5.22 |
| 6 | 401.4 | 2.00 | 7.66 |
| 7 | 468.3 | 2.00 | 12.73 |
| 8 | 535.2 | 0.25 | 49.03 |
| 9 | 267.6 | 1.00 | 53.85 |
| 10 | 133.8 | 1.00 | 52.42 |
| 11 | 0.0 | 1.00 | 50.44 |
| 12 | 133.8 | 1.00 | 52.89 |
| 13 | 267.6 | 1.00 | 54.36 |
| 14 | 401.4 | 1.00 | 57.37 |
| 15 | 427.6 | 2.00 | 85.63 |
| 16 | 468.3 | 0.25 | 115.76 |
| 17 | 401.4 | 1.00 | 116.38 |
| 18 | 267.6 | 1.00 | 115.58 |
| 19 | 133.8 | 1.00 | 113.53 |
| 20 | 0.0 | 12.00 | 106.57 |

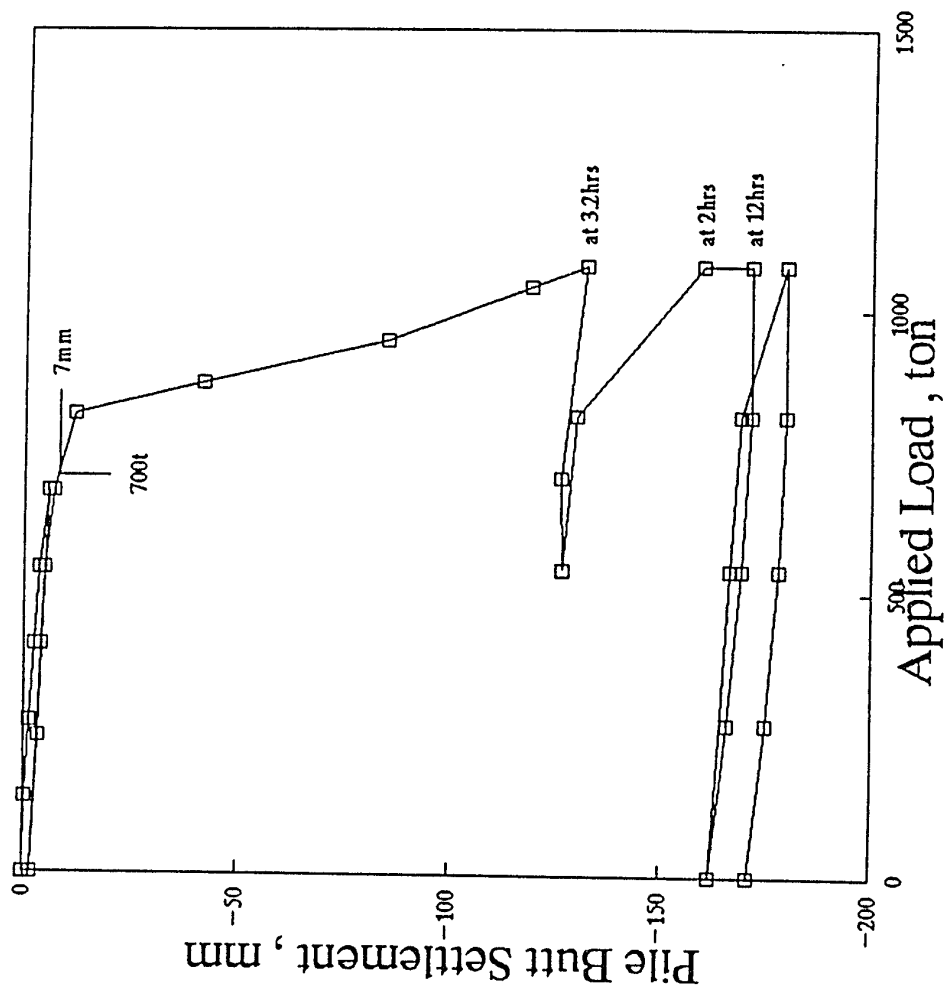
Figure 4.31 Pile Load Test Result of Test Pile No. 8



| Loading Stage | Applied Load, ton | Maximum Duration, hr. | Pile Butt Settlement, mm |
|---------------|-------------------|-----------------------|--------------------------|
| 0 | 0.0 | 0.0 | 0.00 |
| 1 | 66.0 | 1.0 | 0.17 |
| 2 | 132.0 | 1.0 | 0.76 |
| 3 | 198.0 | 1.0 | 1.58 |
| 4 | 264.0 | 1.0 | 2.40 |
| 5 | 330.0 | 2.0 | 3.78 |
| 6 | 396.0 | 2.0 | 5.61 |
| 7 | 462.0 | 2.0 | 7.80 |
| 8 | 528.0 | 2.0 | 13.24 |
| 8 | 528.0 | 12.0 | 16.99 |
| 9 | 396.0 | 1.0 | 15.72 |
| 10 | 264.0 | 1.0 | 14.41 |
| 11 | 132.0 | 1.0 | 12.60 |
| 12 | 0.0 | 12.0 | 10.65 |
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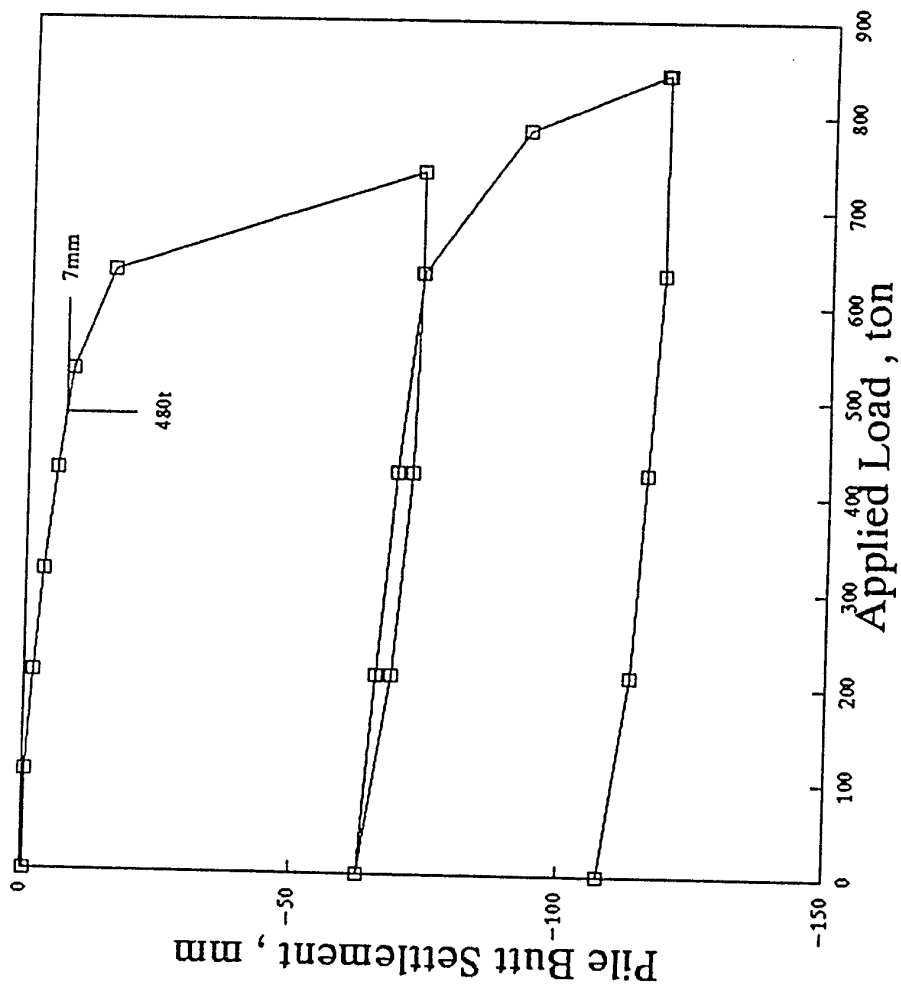
Figure 4.34 Pile Load Test Result of Test Pile No. 11





| Loading Stage | Applied Load, ton | Maximum Duration, hr. | Pile Butt Settlement, mm |
|---------------|-------------------|-----------------------|--------------------------|
| 0 | 0.0 | 0.00 | 0.00 |
| 1 | 135.0 | 1.00 | 0.28 |
| 2 | 270.0 | 1.00 | 1.07 |
| 3 | 405.0 | 2.00 | 2.18 |
| 4 | 540.0 | 2.00 | 3.56 |
| 5 | 675.0 | 0.30 | 5.75 |
| 6 | 243.0 | 1.00 | 3.31 |
| 7 | 0.0 | 1.00 | 1.65 |
| 8 | 405.0 | 1.00 | 3.95 |
| 9 | 540.0 | 1.00 | 4.92 |
| 10 | 675.0 | 2.00 | 6.96 |
| 11 | 810.0 | 2.00 | 11.83 |
| 12 | 869.0 | 0.90 | 42.06 |
| 13 | 945.0 | 2.00 | 84.99 |
| 14 | 1043.5 | 0.10 | 118.82 |
| 15 | 1080.0 | 3.20 | 132.04 |
| 16 | 700.0 | 0.30 | 126.11 |
| 17 | 540.0 | 0.50 | 126.45 |
| 18 | 810.0 | 1.00 | 129.80 |
| 19 | 1080.0 | 2.00 | 159.65 |
| 19 | 1080.0 | 12.00 | 171.42 |
| 20 | 810.0 | 1.00 | 171.17 |
| 21 | 540.0 | 1.00 | 169.14 |
| 22 | 270.0 | 1.00 | 165.73 |
| 23 | 0.0 | 2.00 | 162.00 |
| 24 | 540.0 | 1.00 | 166.36 |
| 25 | 810.0 | 1.00 | 168.93 |
| 26 | 1080.0 | 12.00 | 179.38 |
| 27 | 810.0 | 1.00 | 179.38 |
| 28 | 540.0 | 1.00 | 178.01 |
| 29 | 270.0 | 1.00 | 174.93 |
| 30 | 0.0 | 12.00 | 170.96 |

Figure 4.36 Pile Load Test Result of Test Pile No. 13



| Loading Stage | Applied Load, ton | Maximum Duration, hr. | Pile Butt Settlement, mm |
|---------------|-------------------|-----------------------|--------------------------|
| 0 | 0.0 | 0.0 | 0.00 |
| 1 | 105.5 | 1.0 | 0.67 |
| 2 | 211.0 | 1.0 | 1.86 |
| 3 | 316.5 | 2.7 | 3.39 |
| 4 | 422.0 | 2.0 | 5.62 |
| 5 | 527.5 | 2.0 | 8.32 |
| 6 | 633.0 | 2.0 | 15.61 |
| 7 | 738.5 | 1.0 | 73.07 |
| 8 | 633.0 | 1.0 | 72.97 |
| 9 | 422.0 | 1.0 | 71.88 |
| 10 | 211.0 | 1.0 | 68.55 |
| 11 | 0.0 | 6.0 | 62.85 |
| 12 | 211.0 | 1.0 | 65.72 |
| 13 | 422.0 | 1.0 | 69.07 |
| 14 | 633.0 | 1.0 | 73.37 |
| 15 | 783.5 | 2.0 | 92.75 |
| 16 | 844.0 | 2.0 | 118.25 |
| 16 | 844.0 | 12.0 | 118.81 |
| 17 | 633.0 | 1.0 | 118.47 |
| 18 | 422.0 | 1.0 | 115.87 |
| 19 | 211.0 | 1.0 | 113.11 |
| 20 | 0.0 | 2.0 | 107.76 |

Figure 4.38 Pile Load Test Result of Test Pile No. 15

| Loading Stage | Applied Load, ton | Maximum Duration, hr. | Pile Butt Settlement, mm |
|---------------|-------------------|-----------------------|--------------------------|
| 0 | 0.0 | 0.0 | 0.00 |
| 1 | 132.8 | 1.5 | 0.31 |
| 2 | 265.5 | 1.5 | 1.31 |
| 3 | 398.3 | 1.0 | 2.10 |
| 4 | 531.0 | 1.5 | 4.23 |
| 5 | 664.0 | 2.0 | 8.46 |
| 6 | 796.5 | 2.0 | 16.02 |
| 7 | 929.3 | 2.0 mins. | 19.05 |
| 8 | 0.0 | 12.0 | 14.21 |
| 9 | 265.5 | 1.5 | 15.07 |
| 10 | 531.0 | 1.3 | 17.36 |
| 11 | 796.5 | 1.3 | 19.59 |
| 12 | 929.7 | 2.0 | 23.68 |
| 13 | 1062.0 | 2.0 | 32.54 |
| 13 | 1062.0 | 24.0 | 36.78 |
| 14 | 796.5 | 1.0 | 36.19 |
| 15 | 531.5 | 1.0 | 34.48 |
| 16 | 265.5 | 1.0 | 32.69 |
| 17 | 0.0 | 12.0 | 30.32 |
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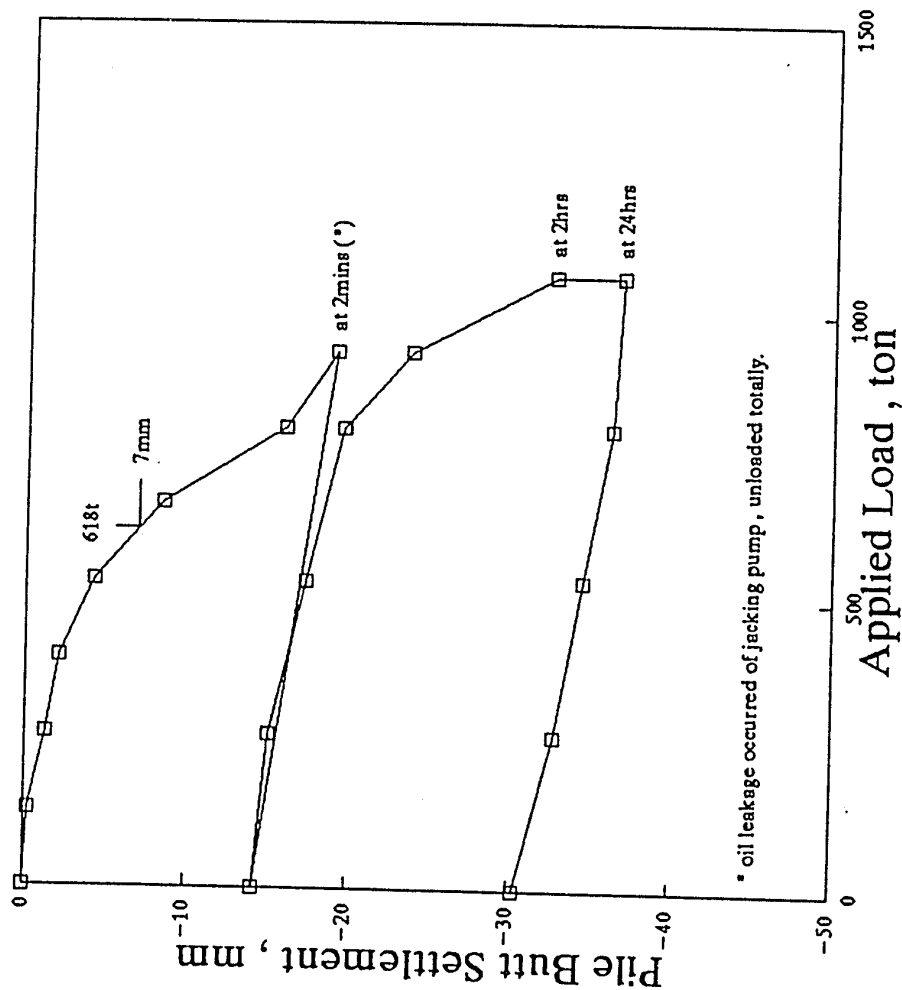


Figure 4.41 Pile Load Test Result of Test Pile No. 18

5.0 diameters center to center.

Loads were generally applied in eight equal increments up to a maximum test load, which was twice the design load, and then unloaded in four equal decrements, as shown in Figures 4.24 through 4.41. Some minor deviation of loading stages occurred as follows:

Pile No. 1 (Figure 4.24): the first loading stage of 73.9 tons was carelessly missed.

Pile No. 4 (Figure 4.27): oil leakage occurred in the jacking pump twice at maximum loading stage. Both unloaded, repaired and reloaded again.

Pile No. 7 (Figure 4.30): the second and third unloading stages of 244.5 and 129.6 tons were carelessly missed.

Pile No. 8 (Figure 4.31): insufficient length of stroke-run of hydraulic jack occurred at maximum loading stage of 535.2 tons, the loading system was reloaded, re-assembled and reloaded again.

Pile No. 13 (Figure 4.36): same as pile No. 8, maximum test load was 1080 tons. An additional loading cycle was applied to see if excessive settlement still occurred.

Pile No. 15 (Figure 4.38): same as pile No. 8, but occurred at

the seventh loading stage of 738.5 tons.

File No. 18 (Figure 4.41): oil leakage occurred at the jacking pump at seventh loading stage of 929.3 tons, unloaded totally, repaired and reloaded again.

Detailed graphical presentations of the load-settlement curves for the pile testings are shown also in Figures 4.24 through 4.41.

4.6 Analysis of Data

The test data of eighteen pile tests are analyzed by comparing the developed shaft frictions determined from the pile test results with the computed values.

For the shaft friction in the clay layer, it is computed based on the two empirical approaches that unit shaft friction of the pile, f , is a function of cone friction, f_s , in the expression as follows:

$$\begin{aligned} f &= \alpha' f_s \quad \text{and} \\ f &= m f_s \end{aligned}$$

where both α and m factors vary with cone friction, decreasing

with an increase in cone friction, as shown in Figures 2.2 and 2.3 which are suggested by Schmertmann (1978) and Tumay et. al. (1981), respectively.

When granular soil is encountered, the relevant shaft friction is computed using the empirical formula suggested by Meyerhof (1976) as:

$$f(T/m^2) = 0.1 N$$

where N = average value of Standard Penetration
Test resistance

The limiting value for shaft friction in granular soils of $12 T/m^2$ usually used by Dutch engineers (Schmertmann, 1978) is applied in the computation.

Tables 4.1 through 4.18 are the computations of the shaft frictions of the eighteen test piles, i.e., piles number 1 to 18. On these tables, the soil parameters used are referred to Figures 4.4 through 4.21, respectively, which are based on the nearest reference boreholes and CPT data from each test pile, and are also based on the construction records of each test pile. For the soil type abbreviations, refer to Table 3.1.

The determined maximum shaft friction of each test pile is defined from the relevant load versus butt settlement curve (Figures 4.24 through 4.41). That is, the load at butt settlement equals to 7.0 mm.

4.7 Discussion of Results

Table 4.19 summarizes the analysis of shaft friction of piles by comparing the computed shaft frictions using equations suggested by Schmertmann (1978) and by Tumay et. al. (1981) and the shaft friction determined from load testings. The computed shaft frictions using equation suggested by Schmertmann are found to be 19~50% lower than the shaft friction determined from load testings, while the computed shaft frictions using equation suggested by Tumay et. al., are found to agree reasonably well within 1~22% (mostly under 18%) from the shaft frictions determined from load testing.

The difference of the two computed shaft frictions is due to the significant difference of α' and m factors for cone friction, f_s , lower than 0.36 kg/cm^2 , as shown in Figure 4.42. The f_s values for the top 20.0 m depth of cohesive soils, where the eighteen test piles located, are generally lower than 0.36 kg/cm^2 .

As a reference, Table 4.19 shows also the percentage of the calculated granular frictions, which were calculated using equation suggested by Meyerhof (1976), from maximum calculated frictions. It suggests that the amount of the granular portions does not significantly influence the analysis of the data, and even if, would still be on the safe side.

Table 4.1

**Shaft Frictional Capacity Calculation of
Test Pile No. 1**

| | | |
|---|------------------------------|---------------------------------|
| Ref. Contract No. : 204A | Pile Dia., d : 1.00 m | Pile Top EL. : +101.80 m |
| Ref. Test Pile No. : 1831-3 (R19-R20) | Pile Buried Length : 41.52 m | Pile Tip EL. : + 58.02 m |
| Ref. Borehole No. : V-31 (1831) (EL.+104.90m) | Max. Test Load : 591.00 t | Const. Date : 04/09/91 |
| Ref. CPT No. : R-B-D03 (EL.+104.80m) | Max. Butt Settl. : 14.44 mm | Test Date : 06/22/91 - 06/24/91 |

| Layer No. | Soil Type | Depth (m) | ΔH (m) | Surface Area, $A = \Delta H \cdot d \cdot \pi$ (m ²) | Ave. Cone Friction, f_s (ksc) | N-Value | Friction of Cohesive Soil | | | Friction of Granular Soil | |
|-------------------------------|-----------|---------------|----------------|--|---------------------------------|---------|---------------------------|-----------|---|---|---|
| | | | | | | | m | α' | Tumay $F1 = f_s \cdot m \cdot A \cdot 10$ (t) | Schmertmann $F2 = f_s \cdot \alpha' \cdot A \cdot 10$ (t) | Meyerhof $F3 = 0.1 \cdot N \cdot A$ (t) |
| 1 | CL | 0.00 - 5.60 | 5.60 | 17.59 | 0.22 | | 1.86 | 1.008 | 71.99 | 39.01 | |
| 2 | SP | 5.60 - 8.30 | 2.70 | 8.48 | | 10 | | | | | |
| 3 | CL | 8.30 - 18.80 | 10.50 | 32.99 | 0.24 | | 1.62 | 0.989 | 128.25 | 78.30 | 8.48 |
| 4 | CL | 18.80 - 33.84 | 15.04 | 47.25 | 0.43 | | 0.71 | 0.816 | 144.25 | 165.79 | |
| 5 | Tf | 33.84 - 41.52 | 7.68 | 24.13 | | 18 | | | | | 43.43 |
| Sub-total Friction (t) | | | | | | | | | $\delta F1 = 344.50$ | $\delta F2 = 283.10$ | $\delta F3 = 51.91$ |

- I. Total Calculated Pile Friction: (a) Tumay's Method, $FT = \delta F1 + \delta F3 = 396$ t
 (b) Schmertmann's Method, $FS = \delta F2 + \delta F3 = 335$ t
- II. Max. Pile Friction from Load Test at Butt Settlement 7 mm, $F7 = 445$ t
- III. Deviation: (a) Tumay's Method, $(F7 - FT)/F7 \cdot 100 = 10.9$ %
 (b) Schmertmann's Method, $(F7 - FS)/FS \cdot 100 = 24.7$ %

Table 4.2

Shaft Frictional Capacity Calculation of
Test Pile No. 2

Ref. Contract No.: 205
 Ref. Test Pile No.: 1839-CP6 (by R24)
 Ref. Borehole No.: R-B-B11 (EL.+104.14m)
 Ref. CPT No.: R-B-D03 (EL.+104.80m)

Pile Dia., d: 1.00 m
 Pile Buried Length: 41.16 m
 Max. Test Load: 752.00 t
 Max. Butt Settl.: 8.91 mm

Pile Top EL.: +103.14 m
 Pile Tip EL.: +60.44 m
 Const. Date: 09/13/90
 Test Date: 12/17/90-12/19/90

| Layer No. | Soil Type | Depth (m) | ΔH (m) | Surface Area, $A=\Delta H \cdot d \cdot \pi$ (m ²) | Ave. Cone Friction, f_s (ksc) | N-Value | Friction of Cohesive Soil | | | | Friction of Granular Soil | |
|------------------------|-----------|---------------|----------------|--|---------------------------------|---------|---------------------------|-----------|---|---|---------------------------------------|------|
| | | | | | | | m | α' | Tumay $F1=f_s \cdot m \cdot A \cdot 10$ (t) | Schmertmann $F2=f_s \cdot \alpha' \cdot A \cdot 10$ (t) | Meyerhof $F3=0.1 \cdot N \cdot A$ (t) | |
| 1 | CL | 0.00 - 7.80 | 7.80 | 24.50 | 0.18 | 10 | 2.46 | 1.049 | 108.51 | 46.27 | | 8.48 |
| 2 | SP | 7.80 - 10.50 | 2.70 | 8.48 | | | | | | | | |
| 3 | CL | 10.50 - 20.80 | 10.30 | 32.36 | 0.20 | | 2.10 | 1.027 | 135.91 | 66.46 | | |
| 4 | CL | 20.80 - 37.10 | 16.30 | 51.21 | 0.46 | | 0.66 | 0.792 | 155.47 | 186.56 | | |
| 5 | CL/ML | 37.10 - 41.16 | 4.06 | 12.75 | 3.00 | | 0.50 | 0.400 | 191.32 | 153.06 | | |
| Sub-total Friction (t) | | | | | | | | | $\delta F1 =$ 591.20 | $\delta F2 =$ 452.35 | $\delta F3 =$ 8.48 | |

I. Total Calculated Pile Friction: (a) Tumay's Method, $FT = \delta F1 + \delta F3 =$ 600 t

(b) Schmertmann's Method, $FS = \delta F2 + \delta F3 =$ 461 t

II. Max. Pile Friction from Load Test at Butt Settlement 7 mm, $F7 =$ 690 t

III. Deviation: (a) Tumay's Method, $(F7 - FT)/F7 \cdot 100 =$ 13.1 %

(b) Schmertmann's Method, $(F7 - FS)/FS \cdot 100 =$ 33.2 %

Table 4.3 Shaft Frictional Capacity Calculation of
Test Pile No. 3

Ref. Contract No.: 205 Pile Dia., d: 1.00 m Pile Top EL.: +104.31 m
 Ref. Test Pile No.: 234 (R21) Pile Buried Length: 42.11 m Pile Tip EL.: +61.20 m
 Ref. Borehole No.: R-B-B20 (EL.+104.07m) Max. Test Load: 717.00 t Const. Date: 04/13/90
 Ref. CPT No.: R-B-D06 (EL.+104.09m) Max. Butt Settl.: 9.92 mm Test Date: 07/17/90-07/19/90

| Layer No. | Soil Type | Depth (m) | ΔH (m) | Surface Area, $A = \Delta H \cdot d \cdot \pi$ (m ²) | Ave. Cone Friction, f_s (ksc) | N-Value | Friction of Cohesive Soil | | | Friction of Granular Soil | |
|------------------------|-----------|---------------|----------------|--|---------------------------------|---------|---------------------------|---|---|--------------------------------|-------|
| | | | | | | | α' | Tumay | Schmertmann | Meyerhof | |
| | | | | | | | m | $F1 = f_s \cdot m \cdot A \cdot 10$ (t) | $F2 = f_s \cdot \alpha' \cdot A \cdot 10$ (t) | $F3 = 0.1 \cdot N \cdot A$ (t) | |
| 1 | MH/OH | 0.00 - 2.70 | 2.70 | 8.48 | 0.16 | | 2.82 | 1.071 | 38.27 | 14.54 | |
| 2 | CL | 2.70 - 17.20 | 14.50 | 45.55 | 0.16 | | 2.82 | 1.071 | 205.54 | 78.06 | |
| 3 | CL | 17.20 - 26.30 | 9.10 | 28.59 | 0.42 | | 0.72 | 0.824 | 86.45 | 98.94 | |
| 4 | CL | 26.30 - 37.00 | 10.70 | 33.62 | 0.68 | | 0.51 | 0.627 | 116.58 | 143.32 | |
| 5 | SM | 37.00 - 40.69 | 3.69 | 11.59 | | 14 | | | | | 16.23 |
| 6 | AND | 40.69 - 42.11 | 1.42 | 4.46 | | 120 | | | | | 53.53 |
| Sub-total Friction (t) | | | | | | | | $\delta F1 =$ | $\delta F2 =$ | $\delta F3 =$ | 69.76 |

- I. Total Calculated Pile Friction: (a) Tumay's Method, $FT = \delta F1 + \delta F3 =$ 517 t
 (b) Schmertmann's Method, $FS = \delta F2 + \delta F3 =$ 405 t
 II. Max. Pile Friction from Load Test at Butt Settlement 7 mm, $F7 =$ 632 t
 III. Deviation: (a) Tumay's Method, $(F7 - FT)/F7 \cdot 100 =$ 18.3 %
 (b) Schmertmann's Method, $(F7 - FS)/FS \cdot 100 =$ 36.0 %

Table 4.4

Shaft Frictional Capacity Calculation of
Test Pile No. 4

Ref. Contract No.: 205
 Ref. Test Pile No.: 110 (R21)
 Ref. Borehole No.: R-B-B19 (EL.+103.81m)
 Ref. CPT No.: R-B-D06 (EL.+104.09m)

Pile Dia., d: 1.50 m
 Pile Buried Length: 41.32 m
 Max. Test Load: 1166.00 t
 Max. Butt Settl.: 18.38 mm

Pile Top EL.: +104.01 m
 Pile Tip EL.: + 61.69 m
 Const. Date: 03/30/90
 Test Date: 06/25/90-06/28/90

| Layer No. | Soil Type | Depth (m) | ΔH (m) | Surface Area, $A = \Delta H \cdot d \cdot \pi$ (m ²) | Ave. Cone Friction, f_s (ksc) | N-Value | Friction of Cohesive Soil | | | Friction of Granular Soil | |
|------------------------|-----------|---------------|----------------|--|---------------------------------|---------|---------------------------|-----------|--|--|---|
| | | | | | | | m | α' | Tumay $F1 = fs \cdot m \cdot A \cdot 10$ (t) | Schmertmann $F2 = fs \cdot \alpha' \cdot A \cdot 10$ (t) | Meyerhof $F3 = 0.1 \cdot N \cdot A$ (t) |
| 1 | CL | 0.00 - 17.00 | 17.00 | 80.11 | 0.16 | | 2.82 | 1.071 | 361.46 | 137.28 | |
| 2 | CL | 17.00 - 26.20 | 9.20 | 43.35 | 0.42 | | 0.72 | 0.824 | 131.10 | 150.04 | |
| 3 | CL | 26.20 - 37.20 | 11.00 | 51.84 | 0.68 | | 0.51 | 0.627 | 179.77 | 221.01 | |
| 4 | SM | 37.20 - 40.33 | 3.13 | 14.75 | | 11 | | | | | 16.22 |
| 5 | AND | 40.33 - 41.32 | 0.99 | 4.67 | | 120 | | | | | 55.98 |
| Sub-total Friction (t) | | | | | | | $\delta F1 = 672.33$ | | | $\delta F2 = 508.33$ | $\delta F3 = 72.21$ |

- I. Total Calculated Pile Friction: (a) Tumay's Method, $FT = \delta F1 + \delta F3 = 745$ t
 (b) Schmertmann's Method, $FS = \delta F2 + \delta F3 = 581$ t
- II. Max. Pile Friction from Load Test at Butt Settlement 7 mm, $F7 = 953$ t
- III. Deviation: (a) Tumay's Method, $(F7 - FT)/F7 \cdot 100 = 21.9$ %
 (b) Schmertmann's Method, $(F7 - FS)/FS \cdot 100 = 39.1$ %

Table 4.5

Shaft Frictional Capacity Calculation of
Test Pile No. 5

Ref. Contract No.: 205
 Ref. Test Pile No.: 2111BP-03 (by R21)
 Ref. Borehole No.: AH-18 (EL. +105.60m)
 Ref. CPT No.: R-B-D07 (EL. +104.85m)

Pile Dia., d: 1.00 m
 Pile Buried Length: 40.90 m
 Max. Test Load: 647.00 t
 Max. Butt Settl.: 8.64 mm

Pile Top EL.: +103.85 m
 Pile Tip EL.: + 61.50 m
 Const. Date: 08/15/90
 Test Date: 10/18/90-10/20/90

| Layer No. | Soil Type | Depth (m) | ΔH (m) | Surface Area, $A = \Delta H \cdot d \cdot \pi$ (m ²) | Ave. Conc Friction, f_s (ksc) | N-Value | Friction of Cohesive Soil | | | Friction of Granular Soil | |
|------------------------|-----------|---------------|----------------|--|---------------------------------|---------|---------------------------|-----------|---|---|---|
| | | | | | | | m | α' | Tumay $F1 = f_s \cdot m \cdot A \cdot 10$ (t) | Schmertmann $F2 = f_s \cdot \alpha' \cdot A \cdot 10$ (t) | Meyerhof $F3 = 0.1 \cdot N \cdot A$ (t) |
| 1 | CL | 0.00 - 14.60 | 14.60 | 45.87 | 0.19 | | 2.28 | 1.038 | 198.70 | 90.46 | |
| 2 | CL | 14.60 - 22.10 | 7.50 | 23.56 | 0.38 | | 0.81 | 0.860 | 72.52 | 77.00 | |
| 3 | CL | 22.10 - 28.70 | 6.60 | 20.73 | 0.60 | | 0.55 | 0.670 | 68.42 | 83.35 | |
| 4 | SM | 28.70 - 37.80 | 9.10 | 28.59 | | 15 | | | | | 42.88 |
| 5 | AND | 37.80 - 40.90 | 3.10 | 9.74 | | 120 | | | | | 116.87 |
| Sub-total Friction (t) | | | | | | | | | $\delta F1 = 339.64$ | $\delta F2 = 250.81$ | $\delta F3 = 159.75$ |

- I. Total Calculated Pile Friction: (a) Tumay's Method, $FT = \delta F1 + \delta F3 = 499$ t
 (b) Schmertmann's Method, $FS = \delta F2 + \delta F3 = 411$ t
- II. Max. Pile Friction from Load Test at Butt Settlement 7 mm, $F7 = 608$ t
- III. Deviation: (a) Tumay's Method, $(F7 - FT)/F7 \cdot 100 = 17.9$ %
 (b) Schmertmann's Method, $(F7 - FS)/FS \cdot 100 = 32.5$ %

Table 4.6 Shaft Frictional Capacity Calculation of
Test Pile No. 6

Ref. Contract No.: 206
 Ref. Test Pile No.: 136 (R24)
 Ref. Borehole No.: AH-22 (EL.+104.98m)
 Ref. CPT No.: R-B-D12 (EL.+105.10m)

Pile Dia., d: 1.50 m
 Pile Buried Length: 19.85 m
 Max. Test Load: 772.00 t
 Max. Butt Settl.: 15.29 mm

Pile Top EL.: +104.07 m
 Pile Tip EL.: + 83.37 m
 Const. Date: 09/07/90
 Test Date: 10/23/90-10/25/90

| Layer No. | Soil Type | Depth (m) | ΔH (m) | Surface Area, $A = \Delta H \cdot d \cdot \pi$ (m ²) | Ave. Cone Friction, f_s (ksc) | N-Value | Friction of Cohesive Soil | | | | Friction of Granular Soil | |
|------------------------|-----------|---------------|----------------|--|---------------------------------|---------|---------------------------|---|---|---|---------------------------|--|
| | | | | | | | α' | Tumay $F1 = f_s \cdot m \cdot A \cdot 10$ (t) | Schmertmann $F2 = f_s \cdot q \cdot A \cdot 10$ (t) | Meyerhof $F3 = 0.1 \cdot N \cdot A$ (t) | | |
| 1 | CL | 0.00 – 10.30 | 10.30 | 48.54 | 0.15 | | 3.00 | 1.082 | 218.42 | 78.78 | | |
| 2 | CL | 10.30 – 13.50 | 3.20 | 15.08 | 0.20 | | 2.10 | 1.027 | 63.33 | 30.97 | | |
| 3 | SM | 13.50 – 18.35 | 4.85 | 22.86 | | 30 | | | | | 68.57 | |
| 4 | SIL | 18.35 – 19.85 | 1.50 | 7.07 | | 120 | | | | | 84.82 | |
| Sub-total Friction (t) | | | | | | | | | $\delta F1 = 281.75$ | $\delta F2 = 109.75$ | $\delta F3 = 153.39$ | |

- I. Total Calculated Pile Friction: (a) Tumay's Method, $FT = \delta F1 + \delta F3 = 435$ t
 (b) Schmertmann's Method, $FS = \delta F2 + \delta F3 = 263$ t
- II. Max. Pile Friction from Load Test at Butt Settlement 7 mm, $F7 = 530$ t
- III. Deviation: (a) Tumay's Method, $(F7 - FT)/F7 \cdot 100 = 17.9$ %
 (b) Schmertmann's Method, $(F7 - FS)/FS \cdot 100 = 50.4$ %

Table 4.7

Shaft Frictional Capacity Calculation of Test Pile No. 7

Ref. Contract No.: 206
 Ref. Test Pile No.: 2311-1 (R23-R24)
 Ref. Borehole No.: AH-20 (EL.+107.15m)
 Ref. CPT No.: R-B-D10 (EL.+107.11m)

Pile Dia., d: 1.00 m
 Pile Buried Length: 34.84 m
 Max. Test Load: 489.00 t
 Max. Butt Settl.: 9.61 mm

Pile Top El.: +105.10 m
 Pile Tip EL.: + 69.66 m
 Const. Date: 06/14/90
 Test Date: 11/13/90-11/15/90

| Layer No. | Soil Type | Depth (m) | ΔH (m) | Surface Area, $A = \Delta H \cdot d \cdot \pi$ (m ²) | Ave. Cone Friction, f_s (ksc) | N - Value | Friction of Cohesive Soil | | | Friction of Granular Soil | |
|------------------------|-----------|---------------|----------------|--|---------------------------------|-----------|---------------------------|-----------|---|---|---|
| | | | | | | | m | α' | Tumay $F1 = f_s \cdot m \cdot A \cdot 10$ (t) | Schmertmann $F2 = f_s \cdot \alpha' \cdot A \cdot 10$ (t) | Meyerhof $F3 = 0.1 \cdot N \cdot A$ (t) |
| 1 | SF | 0.00 - 1.90 | 1.90 | 5.97 | | 10 | 0.68 | 0.800 | 22.90 | 27.14 | 5.97 |
| 2 | CL | 1.90 - 4.30 | 2.40 | 7.54 | 0.45 | | 3.75 | 1.120 | 141.37 | 42.22 | |
| 3 | CL | 4.30 - 14.30 | 10.00 | 31.42 | 0.12 | | 1.36 | 0.964 | 96.90 | 68.69 | |
| 4 | CL | 14.30 - 22.70 | 8.40 | 26.39 | 0.27 | | 0.60 | 0.760 | 35.81 | 45.36 | |
| 5 | CL | 22.70 - 26.50 | 3.80 | 11.94 | 0.50 | | 0.55 | 0.658 | 71.76 | 86.64 | |
| 6 | CL | 26.50 - 33.26 | 6.76 | 21.24 | 0.62 | 120 | | | | | 59.56 |
| 7 | SM-SP | 33.26 - 34.84 | 1.58 | 4.96 | | | | | | | |
| Sub-total Friction (t) | | | | | | | $\delta F1 = 368.75$ | | | $\delta F2 = 270.06$ | $\delta F3 = 65.53$ |

- I. Total Calculated Pile Friction: (a) Tumay's Method, $FT = \delta F1 + \delta F3 = 434$ t
 (b) Schmertmann's Method, $FS = \delta F2 + \delta F3 = 336$ t
- II. Max. Pile Friction from Load Test at Butt Settlement 7 mm, $F7 = 473$ t
- III. Deviation: (a) Tumay's Method, $(F7 - FT)/F7 \cdot 100 = 8.2$ %
 (b) Schmertmann's Method, $(F7 - FS)/FS \cdot 100 = 29.1$ %

Table 4.8

Shaft Frictional Capacity Calculation of
Test Pile No. 8

Ref. Contract No.: 206
 Ref. Test Pile No.: 313 (R22)
 Ref. Borehole No.: R-B-B24 (EL.+105.00m)
 Ref. CPT No.: R-B-B-D08 (EL.+105.21m)

Pile Dia., d: 1.00 m
 Pile Buried Length: 32.26 m
 Max. Test Load: 535.00 t
 Max. Butt Settl.: 55.42 mm

Pile Top El.: +104.99 m
 Pile Tip EL.: + 72.26 m
 Const. Date: 01/23/91 - 01/24/91
 Test Date: 10/16/91 - 10/18/91

| Layer No. | Soil Type | Depth (m) | ΔH (m) | Surface Area, $A = \Delta H \cdot d \cdot \pi$ (m ²) | Ave. Cone Friction, f_s (ksc) | N-Value | Friction of Cohesive Soil | | | Friction of Granular Soil | |
|------------------------|-----------|---------------|----------------|--|---------------------------------|---------|---------------------------|-----------|---|---|---|
| | | | | | | | m | α' | Tumay $F1 = f_s \cdot m \cdot A \cdot 10$ (t) | Schmertmann $F2 = f_s \cdot \alpha' \cdot A \cdot 10$ (t) | Meyerhof $F3 = 0.1 \cdot N \cdot A$ (t) |
| 1 | ML | 0.00 - 0.70 | 0.70 | 2.20 | | 2 | | | | | 0.44 |
| 2 | SM | 0.70 - 4.80 | 4.10 | 12.88 | | 1 | | | | | 1.29 |
| 3 | CL | 4.80 - 10.40 | 5.60 | 17.59 | 0.11 | | 4.00 | 1.132 | 77.41 | 21.91 | |
| 4 | CL | 10.40 - 15.70 | 5.30 | 16.65 | 0.23 | | 1.74 | 0.999 | 66.64 | 38.26 | |
| 5 | CL | 15.70 - 23.40 | 7.70 | 24.19 | 0.36 | | 0.87 | 0.881 | 75.76 | 76.72 | |
| 6 | CL | 23.40 - 29.00 | 5.60 | 17.59 | 0.53 | | 0.59 | 0.736 | 54.55 | 68.63 | |
| 7 | CL | 29.00 - 31.30 | 2.30 | 7.23 | 0.72 | | 0.50 | 0.605 | 26.01 | 31.47 | |
| 8 | AND | 31.30 - 32.30 | 1.00 | 3.14 | | 120 | | | | | 37.70 |
| Sub-total Friction (t) | | | | | | | $\delta F1 = 300.37$ | | | $\delta F2 = 236.99$ | $\delta F3 = 39.43$ |

I. Total Calculated Pile Friction: (a) Tumay's Method, $FT = \delta F1 + \delta F3 = 340$ t

(b) Schmertmann's Method, $FS = \delta F2 + \delta F3 = 276$ t

II. Max. Pile Friction from Load Test at Butt Settlement 7 mm, $F7 = 384$ t

III. Deviation: (a) Tumay's Method, $(F7 - FT)/F7 \cdot 100 = 11.5$ %

(b) Schmertmann's Method, $(F7 - FS)/FS \cdot 100 = 28.0$ %

Table 4.9 Shaft Frictional Capacity Calculation of
Test Pile No. 9

Ref. Contract No.: 206
Ref. Test Pile No.: 226 (R23)
Ref. Borehole No.: R-B-B29 (EL.+107.11m)
Ref. CPT No.: R-B-B-D10 (EL.+107.11m)
Pile Dia., d: 1.20 m
Pile Buried Length: 31.71 m
Max. Test Load: 611.00 t
Max. Butt Settl.: 14.01 mm
Pile Top El.: +104.66 m
Pile Tip EL.: + 72.45 m
Const. Date: 07/22/90-07/25/90
Test Date: 09/30/91-10/02/91

| Layer No. | Soil Type | Depth (m) | ΔH (m) | Surface Area, $A = \Delta H \cdot d \cdot \pi$ (m ²) | Ave. Cone Friction, f_s (ksc) | N-Value | Friction of Cohesive Soil | | | Friction of Granular Soil | |
|------------------------|-----------|---------------|----------------|--|---------------------------------|---------|---------------------------|---|---|---|-------|
| | | | | | | | α' | Tumay $F1 = f_s \cdot m \cdot A \cdot 10$ (t) | Schmertmann $F2 = f_s \cdot \alpha' \cdot A \cdot 10$ (t) | Meyerhof $F3 = 0.1 \cdot N \cdot A$ (t) | |
| 1 | SM | 0.00 - 2.00 | 2.00 | 7.54 | | 18 | | | | | 13.57 |
| 2 | ML | 2.00 - 3.80 | 1.80 | 6.79 | | 1 | | | | | 0.68 |
| 3 | CL | 3.80 - 17.30 | 13.50 | 50.89 | 0.13 | | 1.107 | 231.57 | 73.24 | | |
| 4 | CL | 17.30 - 26.20 | 8.90 | 33.55 | 0.36 | | 0.881 | 105.09 | 106.41 | | |
| 5 | CL | 26.20 - 31.00 | 4.80 | 18.10 | 0.60 | | 0.670 | 59.72 | 72.74 | | |
| 6 | AND | 31.00 - 31.71 | 0.71 | 2.68 | | 120 | | | | | 32.12 |
| Sub-total Friction (t) | | | | | | | | $\delta F1 = 396.37$ | $\delta F2 = 252.40$ | $\delta F3 = 46.37$ | |

I. Total Calculated Pile Friction: (a) Tumay's Method, $FT = \delta F1 + \delta F3 = 443$ t

(b) Schmertmann's Method, $FS = \delta F2 + \delta F3 = 299$ t

II. Max. Pile Friction from Load Test at Butt Settlement 7 mm, $F7 = 513$ t

III. Deviation: (a) Tumay's Method, $(F7 - FT)/F7 \cdot 100 = 13.7$ %

(b) Schmertmann's Method, $(F7 - FS)/FS \cdot 100 = 41.8$ %

Table 4.10 Shaft Frictional Capacity Calculation of
Test Pile No. 10

Ref. Contract No.: 206
 Ref. Test Pile No.: 2209-4 (R22-R23)
 Ref. Borehole No.: AH-19 (EL.+106.59m)
 Ref. CPT No.: R-B-D09 (EL.+106.20m)
 Pile Dia., d: 1.00 m
 Pile Buried Length: 30.70 m
 Max. Test Load: 498.00 t
 Max. Butt Settl.: 15.11 mm
 Pile Top EL.: +103.23 m
 Pile Tip EL.: + 72.08 m
 Const. Date: 07/10/90
 Test Date: 11/20/90-11/22/90

| Layer No. | Soil Type | Depth (m) | ΔH (m) | Surface Area, $A = \Delta H \cdot d \cdot \pi$ (m^2) | Ave. Cone Friction, f_s (ksc) | N-Value | Friction of Cohesive Soil | | | Friction of Granular Soil | |
|------------------------|-----------|---------------|----------------|--|---------------------------------|---------|---------------------------|---|---|---|-------|
| | | | | | | | σ' | Tumay $F1 = f_s \cdot m \cdot A \cdot 10$ (t) | Schmertmann $F2 = f_s \cdot \sigma' \cdot A \cdot 10$ (t) | Meyerhof $F3 = 0.1 \cdot N \cdot A$ (t) | |
| 1 | CL | 0.00 - 11.30 | 11.30 | 35.50 | 0.18 | | 2.46 | 1.049 | 157.19 | 67.03 | |
| 2 | CL | 11.30 - 15.70 | 4.40 | 13.82 | 0.25 | | 1.50 | 0.980 | 51.84 | 33.87 | |
| 3 | CL | 15.70 - 22.60 | 6.90 | 21.68 | 0.35 | | 0.90 | 0.891 | 68.28 | 67.60 | |
| 4 | CL | 22.60 - 26.50 | 3.90 | 12.25 | 0.60 | | 0.55 | 0.670 | 40.43 | 49.25 | |
| 5 | CL | 26.50 - 28.70 | 2.20 | 6.91 | 0.80 | | 0.50 | 0.560 | 27.65 | 30.96 | |
| 6 | SM | 28.70 - 29.70 | 1.00 | 3.14 | | 8 | | | | | 2.51 |
| 7 | SM + GP | 29.70 - 30.70 | 1.00 | 3.14 | | 120 | | | | | 37.70 |
| Sub-total Friction (t) | | | | | | | | $\delta F1 =$ | $\delta F2 =$ | $\delta F3 =$ | 40.21 |
| | | | | | | | | 345.39 | 248.71 | | |

- I. Total Calculated Pile Friction: (a) Tumay's Method, $FT = \delta F1 + \delta F3 =$ 386 t
 (b) Schmertmann's Method, $FS = \delta F2 + \delta F3 =$ 289 t
 II. Max. Pile Friction from Load Test at Butt Settlement 7 mm, $F7 =$ 408 t
 III. Deviation: (a) Tumay's Method, $(F7 - FT)/F7 \cdot 100 =$ 5.5 %
 (b) Schmertmann's Method, $(F7 - FS)/FS \cdot 100 =$ 29.2 %

Table 4.11

Shaft Frictional Capacity Calculation of
Test Pile No. 11

Ref. Contract No.: 206
 Ref. Test Pile No.: 303 (R23)
 Ref. Borehole No.: AH-43 (EL.+106.40m)
 Ref. CPT No.: R-B-D10 (EL.+107.11m)

Pile Dia., d: 1.00 m
 Pile Buried Length: 32.80 m
 Max. Test Load: 528.00 t
 Max. Butt Settl.: 16.99 mm

Pile Top EL.: +104.99 m
 Pile Tip EL.: + 72.20 m
 Const. Date: 12/11/91
 Test Date: 08/23/91 - 08/25/91

| Layer No. | Soil Type | Depth (m) | ΔH (m) | Surface Area, $A = \Delta H \cdot d \cdot \pi$ (m ²) | Ave. Cone Friction, f_s (ksc) | N - Value | Friction of Cohesive Soil | | | Friction of Granular Soil | |
|------------------------|-----------|---------------|----------------|--|---------------------------------|-----------|---------------------------|---|---|---|-------|
| | | | | | | | α' m | Tumay $F1 = f_s \cdot m \cdot A \cdot 10$ (t) | Schmertmann $F2 = f_s \cdot \alpha' \cdot A \cdot 10$ (t) | Meyerhof $F3 = 0.1 \cdot N \cdot A$ (t) | |
| 1 | CL/ML | 0.00 - 17.60 | 17.60 | 55.29 | 0.13 | | 3.50 | 251.58 | 79.57 | | |
| 2 | CL/ML | 17.60 - 26.60 | 9.00 | 28.27 | 0.39 | | 0.78 | 86.01 | 93.73 | | |
| 3 | CL/ML | 26.60 - 31.50 | 4.90 | 15.39 | 0.65 | | 0.53 | 52.53 | 64.04 | | |
| 4 | SM-SP | 31.50 - 32.80 | 1.30 | 4.08 | | 60 | | | | | 24.50 |
| Sub-total Friction (t) | | | | | | | | $\delta F1 = 390.12$ | $\delta F2 = 237.34$ | $\delta F3 =$ | 24.50 |

I. Total Calculated Pile Friction: (a) Tumay's Method, $FT = \delta F1 + \delta F3 = 415$ t

(b) Schmertmann's Method, $FS = \delta F2 + \delta F3 =$

II. Max. Pile Friction from Load Test at Butt Settlement 7 mm, $F7 =$

III. Deviation: (a) Tumay's Method, $(F7 - FT)/F7 \cdot 100 =$

(b) Schmertmann's Method, $(F7 - FS)/FS \cdot 100 =$

415 t

262 t

439 t

5.6 %

40.4 %

Table 4.12 Shaft Frictional Capacity Calculation of
Test Pile No. 12

Ref. Contract No.: 206 Pile Dia., d: 120 mm Pile Top El.: +103.86 m
 Ref. Test Pile No.: 219 (R22) Pile Buried Length: 30.73 m Pile Tip El.: + 72.23 m
 Ref. Borehole No.: R-B-B25 (EL.+105.41m) Max. Test Load: 648.00 t Const. Date: 01/08/91
 Ref. CPT No.: R-B-B-D08 (EL.+105.21m) Max. Butt Settl.: 12.76 mm Test Date: 09/09/91 - 09/11/91

| Layer No. | Soil Type | Depth (m) | ΔH (m) | Surface Area, $A=\Delta H^2 d^2 \pi (m^2)$ | Ave. Cone Friction, f_s (ksc) | N - Value | Friction of Cohesive Soil | | | | Friction of Granular Soil | | | |
|------------------------|-----------|---------------|----------------|--|---------------------------------|-----------|---------------------------|-----------|---------------------------------------|---|---------------------------|------------------------------|---------------|-------|
| | | | | | | | m | α' | Tumay | Schmertmann | Meyerhof | | | |
| | | | | | | | | | $F1=f_s \cdot m \cdot A \cdot 10$ (t) | $F2=f_s \cdot \alpha' \cdot A \cdot 10$ (t) | | $F3=0.1 \cdot N \cdot A$ (t) | | |
| 1 | OH | 0.00 - 5.20 | 5.20 | 19.60 | 0.15 | | 3.00 | 1.082 | 88.22 | 31.82 | | 54.29 | | |
| 2 | CL | 5.20 - 8.90 | 3.70 | 13.95 | 0.11 | | 4.00 | 1.132 | 61.37 | 17.37 | | | | |
| 3 | CL | 8.90 - 13.80 | 4.90 | 18.47 | 0.22 | | 1.86 | 1.008 | 75.59 | 40.96 | | | | |
| 4 | CL | 13.80 - 21.50 | 7.70 | 29.03 | 0.34 | | 0.95 | 0.901 | 93.76 | 88.93 | | | | |
| 5 | CL | 21.50 - 27.60 | 6.10 | 23.00 | 0.54 | | 0.58 | 0.728 | 72.02 | 90.40 | | | | |
| 6 | CL | 27.60 - 29.53 | 1.93 | 7.28 | 0.72 | | 0.50 | 0.605 | 26.19 | 31.69 | | | | |
| 7 | AND | 29.53 - 30.73 | 1.20 | 4.52 | | 120 | | | | | | | | |
| Sub-total Friction (t) | | | | | | | | | $\delta F1 =$ | 417.16 | $\delta F2 =$ | 301.17 | $\delta F3 =$ | 54.29 |

- I. Total Calculated Pile Friction: (a) Tumay's Method, $FT = \delta F1 + \delta F3 = 471$ t
 (b) Schmertmann's Method, $FS = \delta F2 + \delta F3 = 355$ t
 II. Max. Pile Friction from Load Test at Butt Settlement 7 mm, $F7 = 567$ t
 III. Deviation: (a) Tumay's Method, $(F7 - FT)/F7 \cdot 100 = 16.9$ %
 (b) Schmertmann's Method, $(F7 - FS)/FS \cdot 100 = 37.3$ %

Table 4.13 Shaft Frictional Capacity Calculation of
Test Pile No. 13

Ref. Contract No.: 207
 Ref. Test Pile No.: 226 (R25)
 Ref. Borehole No.: R-B-B41 (EL. +102.97m)
 Ref. CPT No.: R-B-D14 (EL. +102.93m)
 Pile Dia., d: 1.20 m
 Pile Buried Length: 39.21 m
 Max. Test Load: 1080.00 t
 Max. Butt Settl.: 170.90 mm
 Pile Top EL.: +102.72 m
 Pile Tip EL.: + 62.10 m
 Const. Date: 08/01/90
 Test Date: 10/22/90 ~ 10/25/90

| Layer | Soil | Depth (m) | ΔH (m) | Surface Area, $A = \Delta H \cdot d \cdot s$ (m^2) | Ave. Cone Friction, f_s (ksc) | N- Value | Friction of Cohesive Soil | | | | Friction of Granular Soil | | | |
|------------------------|------|---------------|-------------------|---|--|-------------|---------------------------|-----------|---|---|---|--------|---------------|-------|
| | | | | | | | m | α' | Tumay $F1 = f_s \cdot m \cdot A \cdot 10^3$ (t) | Schmertmann $F2 = f_s \cdot \alpha' \cdot A \cdot 10^3$ (t) | Meyerhof $F3 = 0.1 \cdot N \cdot A$ (t) | | | |
| | | | | | | | | | | | | | | |
| No. | Type | | | | | | | | | | | | | |
| 1 | OH | 0.00 - 1.50 | 1.50 | 5.65 | 0.15 | | 3.00 | 1.082 | 25.45 | 9.18 | | | | |
| 2 | CL | 1.50 - 5.10 | 3.60 | 13.57 | 0.15 | | 3.00 | 1.082 | 61.07 | 22.03 | | | | |
| 3 | CL | 5.10 - 14.70 | 9.60 | 36.19 | 0.25 | | 1.50 | 0.980 | 135.72 | 88.67 | | | | |
| 4 | CL | 14.70 - 22.90 | 8.20 | 30.91 | 0.33 | | 1.00 | 0.911 | 102.01 | 92.93 | | | | |
| 5 | CL | 22.90 - 32.60 | 9.70 | 36.57 | 0.62 | | 0.55 | 0.658 | 123.56 | 149.18 | | | | |
| 6 | CL | 32.60 - 37.00 | 4.40 | 16.59 | 0.90 | 25 | 0.50 | 0.514 | 74.64 | 76.73 | 8.48 | | | |
| 7 | SM | 37.00 - 37.90 | 0.90 | 3.39 | | 120 | | | | | 59.26 | | | |
| 8 | SM | 37.90 - 39.21 | 1.31 | 4.94 | | | | | | | | | | |
| Sub-total Friction (t) | | | | | | | | | $\delta F1 =$ | 522.46 | $\delta F2 =$ | 438.73 | $\delta F3 =$ | 67.75 |

- I. Total Calculated Pile Friction: (a) Tumay's Method, $FT = \delta F1 + \delta F3 =$ 590 t
 (b) Schmertmann's Method, $FS = \delta F2 + \delta F3 =$ 506 t
 II. Max. Pile Friction from Load Test at Butt Settlement 7 mm, $F7 =$ 700 t
 III. Deviation: (a) Tumay's Method, $(F7 - FT)/F7 \cdot 100 =$ 15.7 %
 (b) Schmertmann's Method, $(F7 - FS)/FS \cdot 100 =$ 27.6 %

Table 4.14 Shaft Frictional Capacity Calculation of
Test Pile No. 14

| | | | | | |
|--|--|------------------------------|--|-------------------------------|--|
| Ref. Contract No. : 207 | | Pile Dia., d : 1.00 m | | Pile Top EL. : +103.43 m | |
| Ref. Test Pile No. : 2608 (by R26) | | Pile Buried Length : 27.69 m | | Pile Tip EL. : + 74.07 m | |
| Ref. Borehole No. : Pilehole 2608 (EL. +105.97m) | | Max. Test Load : 514.00 t | | Const. Date : 06/18/90 | |
| Ref. CPT No. : R-B-D16 (EL. +105.77m) | | Max. Butt Settl. : 9.72 mm | | Test Date : 11/16/90-11/18/90 | |

| Layer | Soil | Depth (m) | ΔH (m) | Surface Area, $A=\Delta H \cdot d^2 \cdot \pi$ (m^2) | Ave. Cone Friction, f_s (ksc) | N = Value | Friction of Cohesive Soil | | | Friction of Granular Soil | |
|------------------------|------|---------------|-------------------|---|--|--------------|---------------------------|---|---|---|--------|
| | | | | | | | α' m | Tumay $F1 = f_s \cdot m \cdot A \cdot 10$ (t) | Schmertmann $F2 = f_s \cdot \alpha' \cdot A \cdot 10$ (t) | Meyerhof $F3 = 0.1 \cdot N \cdot A$ (t) | |
| 1 | ML | 0.00 - 4.70 | 4.70 | 14.77 | 0.12 | | 3.75 | 1.120 | 66.44 | 19.84 | 105.18 |
| 2 | CL | 4.70 - 9.80 | 5.10 | 16.02 | 0.15 | | 3.00 | 1.082 | 72.10 | 26.00 | |
| 3 | CL | 9.80 - 19.20 | 9.40 | 29.53 | 0.21 | | 1.98 | 1.018 | 122.79 | 63.13 | |
| 4 | CL | 19.20 - 23.00 | 3.80 | 11.94 | 0.36 | | 0.87 | 0.881 | 37.39 | 37.86 | |
| 5 | CL | 23.00 - 24.90 | 1.90 | 5.97 | 0.50 | 120 | 0.60 | 0.760 | 17.91 | 22.68 | |
| 6 | AND | 24.90 - 27.69 | 2.79 | 8.77 | | | | | | | |
| Sub-total Friction (t) | | | | | | | $\delta F1 =$ 316.63 | $\delta F2 =$ 169.53 | $\delta F3 =$ 105.18 | | |

I. Total Calculated Pile Friction: (a) Tumay's Method, $FT = \delta F1 + \delta F3 =$ 422 t

II. Max. Pile Friction from Load Test at Butt Settlement 7 mm, $F7 =$ 275 t

III. Deviation: (a) Tumay's Method, $(F7 - FT)/F7 \cdot 100 =$ 507 t

(b) Schmertmann's Method, $(F7 - FS)/FS \cdot 100 =$ 16.8 %

45.8 %

Table 4.15 Shaft Frictional Capacity Calculation of Test Pile No. 15

Ref. Contract No.: 207
 Ref. Test Pile No.: 336 (R25)
 Ref. Borehole No.: R-B-B41 (EL.+102.97m)
 Ref. CPT No.: R-B-D14 (EL.+102.93m)

Pile Dia., d: 1.00 m
 Pile Buried Length: 40.25 m
 Max. Test Load: 844.00 t
 Max. Butt Settl.: 11.04 mm

Pile Top EL.: +104.06 m
 Pile Tip EL.: + 62.21 m
 Const. Date: 08/12/90
 Test Date: 12/22/90-12/24/90

| Layer No. | Soil Type | Depth (m) | ΔH (m) | Surface Area, $A = \Delta H \cdot d \cdot \pi$ (m ²) | Ave. Cone Friction, f_s (ksc) | N-Value | Friction of Cohesive Soil | | | Friction of Granular Soil | |
|------------------------|-----------|---------------|----------------|--|---------------------------------|---------|---------------------------|---|---|---|-------|
| | | | | | | | α' m | Tumay $F1 = f_s \cdot m \cdot A \cdot 10$ (t) | Schmertmann $F2 = f_s \cdot q_c \cdot A \cdot 10$ (t) | Meyerhof $F3 = 0.1 \cdot N \cdot A$ (t) | |
| 1 | OH | 0.00 - 6.10 | 6.10 | 19.16 | 0.15 | | 3.00 | 1.082 | 86.24 | 31.10 | |
| 2 | CL | 6.10 - 15.70 | 9.60 | 30.16 | 0.25 | | 1.50 | 0.980 | 113.10 | 73.89 | |
| 3 | CL | 15.70 - 23.80 | 8.10 | 25.45 | 0.33 | | 1.00 | 0.911 | 83.97 | 76.50 | |
| 4 | CL | 23.80 - 33.60 | 9.80 | 30.79 | 0.62 | | 0.55 | 0.658 | 104.03 | 125.60 | |
| 5 | CL | 33.60 - 37.70 | 4.10 | 12.88 | 0.90 | | 0.50 | 0.514 | 57.96 | 59.59 | |
| 6 | SM | 37.70 - 40.25 | 2.55 | 8.01 | | 25 | | | | | 20.03 |
| Sub-total Friction (t) | | | | | | | | $\delta F1 =$ | $\delta F2 =$ | $\delta F3 =$ | 20.03 |
| | | | | | | | | 445.30 | 366.68 | | |

I. Total Calculated Pile Friction: (a) Tumay's Method, $FT = \delta F1 + \delta F3 = 465$ t

(b) Schmertmann's Method, $FS = \delta F2 + \delta F3 = 387$ t

II. Max. Pile Friction from Load Test at Butt Settlement 7 mm, $F7 = 480$ t

III. Deviation: (a) Tumay's Method, $(F7 - FT)/F7 \cdot 100 = 3.1$ %

(b) Schmertmann's Method, $(F7 - FS)/FS \cdot 100 = 19.4$ %

Table 4.16 Shaft Frictional Capacity Calculation of
Test Pile No. 16.

Ref. Contract No.: 207 Pile Dia., d: 1.00 m Pile Top El.: +102.67 m
 Ref. Test Pile No.: 301 (R25) Pile Buried Length: 40.72 m Pile Tip EL.: + 60.95 m
 Ref. Borehole No.: A-B-B19 (EL.+102.94m) Max. Test Load: 844.00 t Const. Date: 08/08/90
 Ref. CPT No.: R-B-B-D14 (EL.+102.93m) Max. Butt Settl.: 76.90 mm Test Date: 10/11/90-10/13/90

| Layer No. | Soil Type | Depth (m) | ΔH (m) | Surface Area, $A = \Delta H \cdot d \cdot \pi$ (m^2) | Ave. Cone Friction, f_s (ksc) | N-Value | Friction of Cohesive Soil | | | Friction of Granular Soil | |
|------------------------|-----------|---------------|----------------|--|---------------------------------|---------|---------------------------|---|---|---|-------|
| | | | | | | | α' | Tumay $F1 = f_s \cdot m \cdot A \cdot 10$ (t) | Schmertmann $F2 = f_s \cdot \alpha' \cdot A \cdot 10$ (t) | Meyerhof $F3 = 0.1 \cdot N \cdot A$ (t) | |
| 1 | ML | 0.00 - 5.40 | 5.40 | 16.96 | 0.15 | | 1.082 | 76.34 | 27.53 | | |
| 2 | CL | 5.40 - 15.00 | 9.60 | 30.16 | 0.25 | | 0.980 | 113.10 | 73.89 | | |
| 3 | CL | 15.00 - 23.10 | 8.10 | 25.45 | 0.33 | | 0.911 | 83.97 | 76.50 | | |
| 4 | CL | 23.10 - 33.00 | 9.90 | 31.10 | 0.62 | | 0.658 | 105.09 | 126.88 | | |
| 5 | CL | 33.00 - 38.10 | 5.10 | 16.02 | 0.90 | | 0.514 | 72.10 | 74.12 | | |
| 6 | AND | 38.10 - 40.72 | 2.62 | 8.23 | | 77 | | | | | 63.38 |
| Sub-total Friction (t) | | | | | | | | $\delta F1 = 450.61$ | $\delta F2 = 378.93$ | $\delta F3 =$ | 63.38 |

I. Total Calculated Pile Friction: (a) Tumay's Method, $FT = \delta F1 + \delta F3 = 514$ t

(b) Schmertmann's Method, $FS = \delta F2 + \delta F3 = 442$ t

II. Max. Pile Friction from Load Test at Butt Settlement 7 mm, $F7 = 572$ t

III. Deviation: (a) Tumay's Method, $(F7 - FT)/F7 \cdot 100 = 10.1$ %

(b) Schmertmann's Method, $(F7 - FS)/FS \cdot 100 = 22.7$ %

Table 4.17 Shaft Frictional Capacity Calculation of
Test Pile No. 17

Ref Contract No.: 207 Pile Dia., d: 1.20 m Pile Top EL.: +104.00 m
 Ref Test Pile No.: 257 (R26) Pile Buried Length: 28.14 m Pile Tip EL.: + 75.52 m
 Ref Borehole No.: R-B-B46 (EL.+106.12m) Max. Test Load: 716.00 t Const. Date: 03/22/90
 Ref CPT No.: R-B-B-D15 (EL.+104.70m) Max. Butt Settl.: 38.57 mm Test Date: 06/15/90-06/17/90

| Layer No. | Soil Type | Depth (m) | ΔH (m) | Surface Area, $A = \Delta H \cdot d \cdot \pi$ (m ²) | Ave. Cone Friction, f_s (ksc) | N - Value | Friction of Cohesive Soil | | | | Friction of Granular Soil | |
|------------------------|-----------|---------------|----------------|--|---------------------------------|-----------|---------------------------|---|---|---|---------------------------|--|
| | | | | | | | σ' (t) | Tumay $F1 = f_s \cdot m \cdot A \cdot 10$ (t) | Schmertmann $F2 = f_s \cdot \sigma' \cdot A \cdot 10$ (t) | Meyerhof $F3 = 0.1 \cdot N \cdot A$ (t) | | |
| 1 | CH/CL | 0.00 - 3.70 | 3.70 | 13.95 | 0.11 | | 4.00 | 61.37 | 17.37 | | 54.29 | |
| 2 | CL | 3.70 - 16.50 | 12.80 | 48.25 | 0.16 | | 2.82 | 217.73 | 82.69 | | | |
| 3 | CL | 16.50 - 26.94 | 10.44 | 39.36 | 0.38 | | 0.81 | 121.14 | 128.62 | | | |
| 4 | AND | 26.94 - 28.14 | 1.20 | 4.52 | | 120 | | | | | | |
| Sub-total Friction (t) | | | | | | | | $\delta F1 = 400.24$ | $\delta F2 = 228.68$ | $\delta F3 = 54.29$ | | |

I. Total Calculated Pile Friction: (a) Tumay's Method, $FT = \delta F1 + \delta F3 = 455$ t

(b) Schmertmann's Method, $FS = \delta F2 + \delta F3 = 283$ t

II. Max. Pile Friction from Load Test at Butt Settlement 7 mm, $F7 = 473$ t

III. Deviation: (a) Tumay's Method, $(F7 - FT)/F7 \cdot 100 = 3.9$ %

(b) Schmertmann's Method, $(F7 - FS)/FS \cdot 100 = 40.2$ %

Table 4.18 Shaft Frictional Capacity Calculation of
Test Pile No. 18

Ref. Contract No.: 207 Pile Dia., d: 1.50 m Pile Top EL.: +105.26 m
 Ref. Test Pile No.: 142 (R26) Pile Buried Length: 30.14 m Pile Tip EL.: + 73.82 m
 Ref. Borehole No.: R-B-B45 (EL.+105.36m) Max. Test Load: 1062.00 t Const. Date: 04/19/90
 Ref. CPT No.: R-B-D15 (EL.+104.70m) Max. Butt Settl.: 36.78 mm Test Date: 07/11/90 - 07/14/90

| Layer | Soil | Depth (m) | ΔH (m) | Surface Area, $A=\Delta H \cdot d \cdot \pi$ (m^2) | Ave. Cone Friction, f_s (ksc) | N - Value | Friction of Cohesive Soil | | | | Friction of Granular Soil | | | | |
|------------------------|------|---------------|-------------------|---|--|------------------|---------------------------|------------------|---|---|---|--|----------------------|----------------------|----------------------|
| | | | | | | | m | σ' (t) | Tumay $F1=f_s \cdot m \cdot A \cdot 10$ (t) | Schmertmann $F2=f_s \cdot \sigma' \cdot A \cdot 10$ (t) | Meyerhof $F3=0.1 \cdot N \cdot A$ (t) | | | | |
| No. | Type | | | | | | | | | | | | | | |
| 1 | OH | 0.00 - 5.30 | 5.30 | 24.98 | 0.11 | | 4.00 | 1.132 | 109.89 | 31.10 | | | | | |
| 2 | CL | 5.30 - 17.00 | 11.70 | 55.13 | 0.16 | | 2.82 | 1.071 | 248.77 | 94.48 | | | | | |
| 3 | CL | 17.00 - 26.60 | 9.60 | 45.24 | 0.38 | | 0.81 | 0.860 | 139.25 | 147.84 | | | | | |
| 4 | SM | 26.60 - 29.25 | 2.65 | 12.49 | | 50 | | | | | 62.44 | | | | |
| 5 | SM | 29.25 - 30.14 | 0.89 | 4.19 | | 120 | | | | | 50.33 | | | | |
| Sub-total Friction (t) | | | | | | | | | | | | | $\delta F1 =$ 497.91 | $\delta F2 =$ 273.42 | $\delta F3 =$ 112.77 |

- I. Total Calculated Pile Friction: (a) Tumay's Method, $FT = \delta F1 + \delta F3 =$ 611 t
 (b) Schmertmann's Method, $FS = \delta F2 + \delta F3 =$ 386 t
 II. Max. Pile Friction from Load Test at Butt Settlement 7 mm, $F7 =$ 618 t
 III. Deviation: (a) Tumay's Method, $(F7 - FT)/F7 \cdot 100 =$ 1.2 %
 (b) Schmertmann's Method, $(F7 - FS)/FS \cdot 100 =$ 37.5 %

Table 4.19 Analysis of Frictional Capacity

| Pile No. | Pile Dia., m | Pile Buried Length, m | Maximum Pile Friction, Ton | | | Prediction Error *, % | | Percentage of Calculated Granular Friction from Max. Friction, % | |
|----------|--------------|-----------------------|----------------------------|---------------------------------|---------------------------------------|-----------------------|----------------------|--|----------------------|
| | | | Determined from Load Test | Calculated using Tumay's Method | Calculated using Schmertmann's Method | Tumay's Method | Schmertmann's Method | Tumay's Method | Schmertmann's Method |
| | | | | | | | | | |
| 1 | 1.00 | 41.52 | 445 | 396 | 335 | 10.9 | 24.7 | 13.1 | 15.5 |
| 2 | 1.00 | 41.16 | 690 | 600 | 461 | 13.1 | 33.2 | 1.4 | 1.8 |
| 3 | 1.00 | 42.11 | 632 | 517 | 405 | 18.3 | 36 | 13.5 | 17.2 |
| 4 | 1.50 | 41.32 | 953 | 745 | 581 | 21.9 | 39.1 | 9.7 | 12.4 |
| 5 | 1.00 | 40.9 | 608 | 499 | 411 | 17.9 | 32.5 | 32 | 38.9 |
| 6 | 1.50 | 19.85 | 530 | 435 | 263 | 17.9 | 50.4 | 35.3 | 58.3 |
| 7 | 1.00 | 34.84 | 473 | 434 | 336 | 8.2 | 29.1 | 15.1 | 19.5 |
| 8 | 1.00 | 32.26 | 384 | 340 | 276 | 11.5 | 28 | 11.6 | 14.3 |
| 9 | 1.20 | 31.71 | 513 | 443 | 299 | 13.7 | 41.8 | 10.5 | 15.5 |
| 10 | 1.00 | 30.7 | 408 | 386 | 289 | 5.5 | 29.2 | 10.4 | 13.9 |
| 11 | 1.00 | 32.8 | 439 | 415 | 262 | 5.6 | 40.4 | 5.9 | 9.4 |
| 12 | 1.20 | 30.73 | 567 | 471 | 355 | 16.9 | 37.3 | 11.5 | 15.3 |
| 13 | 1.20 | 39.21 | 700 | 590 | 506 | 15.7 | 27.6 | 11.5 | 13.4 |
| 14 | 1.00 | 27.69 | 507 | 422 | 275 | 16.8 | 45.8 | 24.9 | 38.2 |
| 15 | 1.00 | 40.25 | 480 | 465 | 387 | 3.1 | 19.4 | 4.3 | 5.2 |
| 16 | 1.00 | 40.72 | 572 | 514 | 442 | 10.1 | 22.7 | 12.3 | 14.3 |
| 17 | 1.20 | 28.14 | 473 | 455 | 283 | 3.9 | 40.2 | 11.9 | 19.2 |
| 18 | 1.50 | 30.14 | 618 | 611 | 386 | 1.2 | 37.5 | 18.5 | 29.2 |

* Prediction Error = (Determined - Calculated) / Determined x 100%

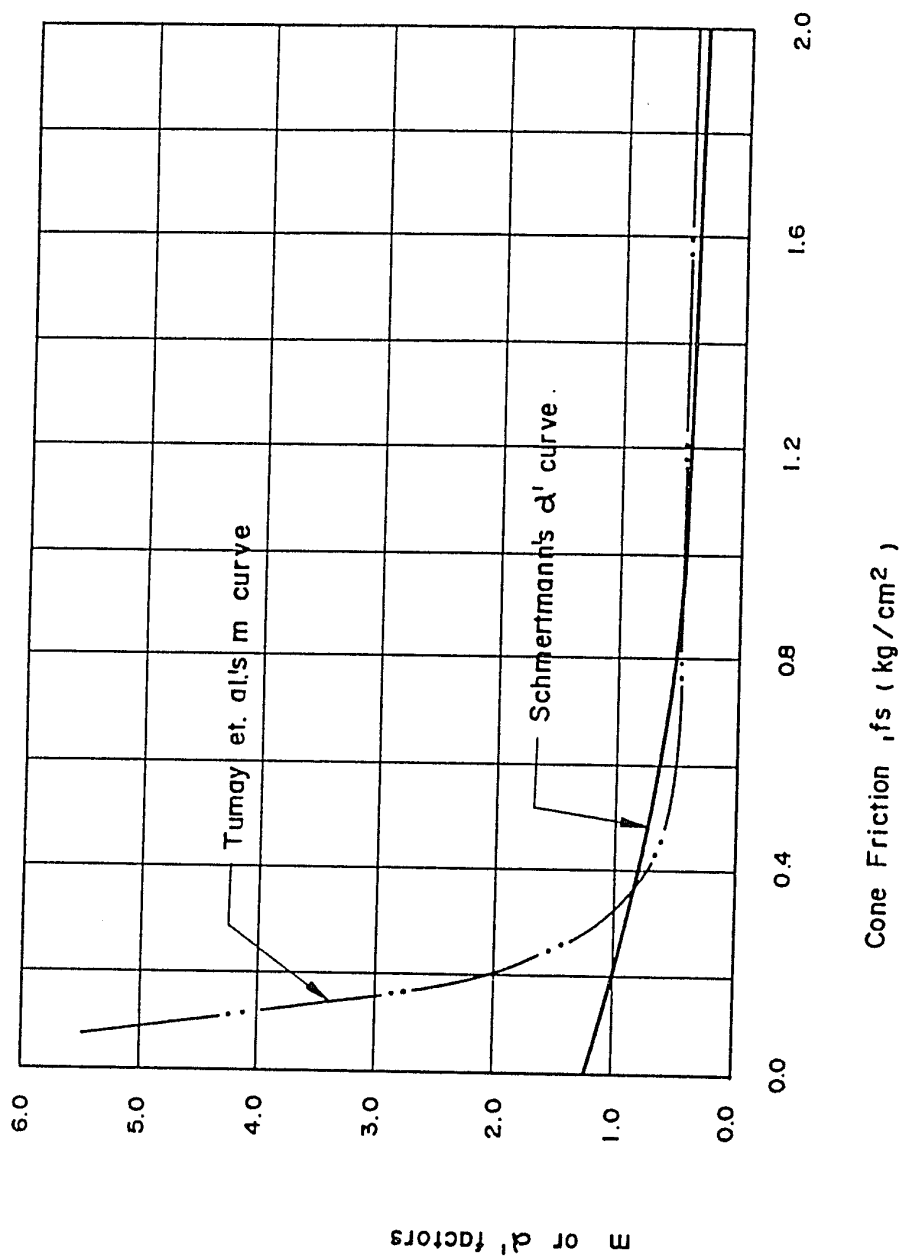


Figure 4.42 Comparison of α' and m factors

CHAPTER V

CONCLUSIONS

1. The available instrumented test piles data back up that the maximum shaft friction of a test pile in cohesive soils can be defined from its load versus butt settlement curve, that is, the load at butt settlement equals to 7.0 mm.
2. The analysis of the results of eighteen pile load tests located within the tributary area of the Keelung River, which predominately consists of fine grained silts and clays, suggests that the present state-of-the art methods for utilizing Cone Penetration Test (CPT) data for maximum shaft friction prediction in cohesive soils agree reasonably well when using method suggested by Tumay et. al. (1981), i.e., within 1-22% with mostly under 18% and on the safe side, compared with the observed values from load testings; but the method suggested by Schmertmann (1978) tends to underestimate the load carrying ability (up to 50%) of the piles in this region.

3. The difference of the two computed shaft frictions is due to the significant difference of α' and m factors for cone friction lower than 0.36 kg/cm^2 as shown in Figure 4.42, where the m factor is higher than the α' factor.
4. In general, the interpretive procedure using CPT data has the following advantages that seem to warrant its application:
 - a. The procedure currently requires only the most popular in-situ tests data, i.e., Cone Penetration Test (CPT) and Standard Penetration Test (SPT), which are relatively inexpensive and easy to implement.
 - b. The interpretive technique does not depend on a detailed knowledge of pile properties or of soil-pile stress transfer throughout the length of the pile.

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